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DESIGN AND PERFORMANCE
OF TAILINGS DAMS

by

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A THESIS

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled "DESIGN AND PERFORMANCE OF TAILINGS DAMS" submitted by Hari Krishan Mittal in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

To The Memory of My
Dear Departed Mother

ABSTRACT

Tailings embankments have been constructed, in the past, with little or no regard to some of the basic principles of dam engineering. This has resulted in many failures, some with disastrous consequences. These failures, along with the growing concern for pollution control, have sparked an unprecedented interest in the safety of waste embankments.

This thesis is intended to synthesize design criteria for the construction of tailings embankments. The general approach followed in this undertaking has been to rationalize the requirements for safe construction in that possible wastefulness of overdesign may be avoided.

The thesis outlines operating conditions under which a safe tailings embankment can be built by the economical upstream method of construction. High tailings dams, however, must be built by the downstream techniques or by one of the modified methods presented in this thesis. The viability of these methods would, however, depend upon the sand recovery from tailings. It is shown that through the judicious handling of materials, a fairly high degree of

sand recovery can be achieved since a much higher percentage of fines can be tolerated in the sand than that presently specified at most projects.

The existing state of under-consolidation of slimes is shown to effect the rate of seepage through a dam and the other related features in the design of tailings dams. New methods of measuring the dominant parameters of in situ density and in situ permeability are explored and suitable equipment and techniques have been developed. In situ density is measured by a nuclear probe and in situ permeability above the water table is deduced from a constant head infiltration test. Both tests have been applied under field conditions and shown to give reliable results. The detailed laboratory testing and the field investigations at three typical tailings dams have provided a considerable insight into the requirements of materials handling in the construction of a safe tailings embankment.

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CHAPTER I

INTRODUCTION

1.1 Tailings and the Significance of Their Disposal

Tailings are a waste product of the mining industry. In a typical mining operation, the recovery of valuable minerals is achieved by crushing and grinding ore to silt and sand sizes, and then subjecting the ground ore to a process of "concentration". Although there are several other techniques, "flotation" is the most commonly used concentration process today and perhaps has been since its introduction to the mining industry in 1924 (Encyclopedia Britannica). This process consists of treating the ground ore in a bubbling mixture of water and suitable chemicals so that the metallic minerals adhere to air bubbles which float to the surface and are removed as a concentrate in the froth. The remaining ground rock constitutes the unusable waste material, commonly known as tailings.

The volume of tailings to be discarded depends on the type of ore and the mining operation. In the case of the metallic minerals, ores containing less than one percent marketable minerals are being mined at the present time.

The new open pit mines in these low-grade ores and the tar sands plants, in northern Alberta, are among the largest producers of tailings. The future oil shale developments in the United States, also, are likely to produce some very large volumes of tailings.

In view of the large volumes of tailings produced in these operations, the cost of tailings disposal can be a significant portion of the total capital outlay. For example, Tetu and Pells (1971) report that at Brenda Mines in British Columbia, of the initial capital outlay of 62.3 million dollars required to put the mine into operation, 8.2 percent (or 5.1 million dollars) was allocated to the tailings system. It is further estimated that the on-going cost of the disposal facilities at this project is likely to be in the range of 5 to 10 cents per ton of tailings disposed. With the total amount of tailings estimated to be in the order of 175 to 200 million tons during the operating life of the mine, this will result in a significant expense in addition to the initial 5.1 million dollars. Because of the high costs involved, the very viability of the more marginal mining properties can, therefore, depend on the economical disposal of tailings.

Disposal of tailings presents a major potential threat to the environment as it may result in the possible pollution of surrounding streams and ground water system.

The growing awareness of the public for the protection of the environment is resulting in stringent regulations to control the disposal of industrial wastes. Under these circumstances, a mining operator has to provide a design for the disposal of tailings which is not only safe in the conventional engineering sense, but also pollution free, before permission may be forthcoming for the development of a mining property.

1.2 Functions and a Brief History of Tailings Dams

Although the method of tailings disposal followed at a few mines involves no more than disposing of tailings into the nearest stream or body of water, more generally, dykes (or dams) are constructed to create storage areas for the disposal of tailings. These "tailings dams" also serve to provide initial storage of water for mill start-up and temporary storage for water to be clarified for reuse in the flotation process.

Although it is conceivable that some form of tailings disposal must have been used throughout history, construction of the earliest tailings dams (as we know them today) reported in engineering literature appears to have been started in the 1920's. At the present time (April, 1974), a review of available engineering literature on tailings dams or similar embankments for retaining other solid wastes, indicates that they are being constructed in

many industries all over the world. These include, for example, gold mining in South Africa; mining for base metals in South America, Africa, the United States and Canada; coal mining in many countries; phosphate mines in Florida; china clay wastes in southwest England (Ripley, 1973); general industry in the storage of chemical waste; and thermal plants for storage of ash refuse, to name a few.

1.3 Past Performance of Tailings Dams

As tailings are a waste product, their disposal is a direct cost to the project without any "dollar benefit". It is generally believed that the overriding requirement in building tailings dams in the past has been to dispose of tailings as cheaply as possible and that the construction techniques followed are a reflection of this overriding criterion, with little or no consideration for proper engineering practice.

Consequently, the past performance of tailings dams has been anything but encouraging. In fact, the mining industry has been plagued with failures of tailings dams. Hoare (1972) reports that:

> "Major failures have occurred in Canada, United States, Chile, Australia, New Zealand, South Africa, England and throughout the mining countries of Europe."

Typical of the more serious of these failures are the Barahona tailings dam in 1928 and the El Cobre tailings dams in 1965 in Chile (Dobry and Alvarez, 1967), and more

recently the Buffalo Creek coal refuse dam in 1972 in the United States (Osborn, 1973).

Strictly speaking, failure of the Aberfan waste dump in Britain (Aberfan, 1967) cannot be classed as a tailings dam failure. It is generally believed, nevertheless, that it is the Aberfan disaster, in which over 100 school children were killed, that shocked the western world out of its apathy to recognize the potential dangers to life and property presented by these improperly designed waste embankments.

In 1968, following the Aberfan disaster, the Canadian government initiated a study on the stability of waste embankments under the auspices of the Canadian Advisory Committee on Rock Mechanics (1969). Answers to a questionnaire sent to the mining companies, as part of this study, indicate that 35 percent of the companies have had stability problems with their tailings dams. The problems reported are primarily in the category of serious failures. Investigations by Hoare (1972) and his discussions with technical personnel on numerous mining operations, however, indicate that over 90 percent of the larger mining operations have had tailings dam failures of some kind. These failures vary from minor slip failures with little resulting damage to catastrophic failures.

In spite of these large number of failures experienced by the mining industry, there has been only a small number of failures to tailings dams actually reported in the engineering literature, in any detail. Hoare (1972) points out that:

> "...... the majority of the details relating to the failure of tailings dams are withheld by mining officials to protect their interests and to reduce the embarrassment of the companies."

Table 1.1 includes most of the reported cases of failures of tailings dams.

1.4 Mining in Canada - Present and Future

The mining industry in Canada constitutes an important part of the national economy. Latest figures available from Statistics Canada indicate that revenue from mineral production (including petroleum and natural gas) reached an impressive figure of over 5.9 billion dollars in 1971, which is an increase of over one billion dollars from 1969 and more than double the revenue in 1962. Fuels accounted for about 30 percent of the revenue, metallic minerals 54 percent, and the non-metallics and structural materials the remaining 16 percent.

Based on the work of Hoare (1972), it is estimated that the quantity of tailings produced by the Canadian mining industry during 1971 exceeded 400,000,000 tons. In terms of tailings produced, iron ore, and copper and nickel

based ores are the leaders. It appears that the mining of iron ore in Canada has reached a state of maturity and that a larger number of new mines will likely be developed in nickel and copper based ores with particular emphasis on the copper.

In recent years, the advances in technology and mining equipment, and increasingly higher prices of metals on the world market has shifted the emphasis from the small underground operations to large low-grade open pit mines. For example, as recently as 1969, there were only three operating mines in British Columbia with throughput capacity in excess of 15,000 tons per day. Since 1969, however, at least six more large open pit mines have been put into production with the largest one at a throughput capacity of about 38,000 tons per day and several more are at various stages of planning.

The present forecasts indicate that production of nickel and copper in the Canadian mining industry will continue to increase to claim its full share of the ever increasing world demand and that most of the growth in production will come from large low-grade open pit mines. It is anticipated that the new large nickel mines will be, essentially, located in Ontario and Manitoba, and the copper mines in British Columbia.

Typically, each of these large open pit mines will produce tailings in excess of 200 million tons requiring some exceptionally high tailings dams. Several tailings dams being planned at the present time or envisaged for the future will be in excess of 500 feet in height.

1.5 Recommendations of the Sub-Committee on Stability of Waste Embankments and Reason for This Investigation

In 1968, a study was undertaken by the Sub-committee on Stability of Waste Embankments under the authority of the Canadian Advisory Committee on Rock Mechanics. The general goals of the study were:

i) to define the problems of stability of tailings dams, waste dumps and ore piles;

ii) to determine existing controls and legislation in Canada; and

iii) to develop recommendations for design guides, education, legislation and research in Canada.

Although all the details of the study are covered by the Canadian Advisory Committee on Rock Mechanics (1969), a brief outline of the observations and the recommendations made by the Sub-committee relevant to tailings dams is considered of interest here and is presented below.

1.5.1 Observations

i) The literature review indicated that many serious failures of waste embankments have occurred throughout the world, including some in Canada.

ii) Based on the results of a questionnaire, approximately 35% of tailings dams constructed by mining companies in Canada have suffered some degree of instability.

iii) Stability investigations were only performed for 26% of the tailings dams reported.

iv) Present mining regulations in Canada generally do not require a detailed evaluation of stability prior to construction.

v) Ninety-four percent of the mining engineers who submitted completed questionnaires indicated that it is desirable to establish definite guidelines for the design and construction of waste embankments.

1.5.2 Recommendations

In view of the above observations, a brief outline of the recommendations made by the Sub-committee follows:

i) a design guide for the investigation, design, and construction of waste embankments should be developed;

ii) education programs through universities (both in extension and undergraduate courses) and through the Canadian Institute of Mining and Metallurgy, which deal with the basic considerations of stability should be encouraged;

iii) the provincial governments should be informed of the current practices in connection with construction of waste embankments, of the value of specialists trained in stability engineering in appraising design and of the value of periodic inspection of major waste structures; and

iv) new programs should be established to include practical research relating to site investigations, design, construction, maintenance and inspection of waste embankments.

1.5.3 Reason for This Investigation

In accordance with the Sub-committee's recommendations, the following steps have already been taken:

i) the Department of Energy, Mines and Resources (1972) has prepared a "Tentative Design Guide for Waste Embankments in Canada";

ii) several universities, including the University of Alberta, have held workshops on the design of tailings embankments for representatives

of the mining industry at large; and

iii) several provincial governments are preparing regulations to control the design and construction of tailings embankments as discussed in the next section (1.6).

Following the general intent of the Sub-committee's fourth recommendation, the present investigation has been undertaken to carry out practical research in soil mechanics relevant to design and materials handling for the construction of economical but safe tailings dams. Some research work has already been done by Hoare (1972). It is felt that although Hoare's work is likely to be of value in planning tailings disposal systems, it provides only a limited amount of soil mechanics input into the design of tailings dams and hence, forms only a general but certainly a valuable background for this work. Pertinent aspects of Hoare's work will be discussed in detail and incorporated into this study where applicable.

1.6 Legislation and Environmental Requirements

There are two aspects of tailings dams that involve public concern. One, that involves a failure of a tailings dam in which large volumes of water and semi-fluid tailings are released which may not only cause extensive downstream pollution but also pose a serious threat to life and property; the other, that relates to the

possibility of pollution, under normal operation, in which effluent may escape through or around a tailings dam and pollute streams or ground water system of the surrounding area. In recent years, pollution control has become a very topical and contentious issue with the public. Consequently, under increasing public demands, there has been a trend towards stricter regulations affecting all aspects of design, construction and operation of tailings dams.

One of the observations reported by the Sub-committee on Waste Disposal, as discussed in Section 1.5.1, was that the existing mining regulations, at that time, did not require a detailed evaluation of stability prior to construction of tailings dams - much less anti-pollution measures. Since then, however, governmental regulatory agencies across Canada have been moving with haste to establish appropriate control regulations. For example, in British Columbia, some very strict regulations were established in 1971 (Klohn, 1972B), which are considered to be typical for future mines in Canada. A copy of the above regulations issued under the authority of the Chief Inspector of Mines, in British Columbia, is presented in in Appendix A. Appropriate steps are also being taken in the United States to establish similar regulations (Osborn, 1973).

1.7 Historical Review of Interest in Tailings Dams

Detailed considerations of literature on previous studies, investigations and observations relevant to various aspects of tailings dams will be dealt with in appropriate sections of the thesis. Only a brief discussion is presented at this time to trace the historical development of engineering interest and involvement of the geotechnical profession in the design and construction of tailings dams.

Until recently, most papers reported in engineering literature had been authored by the supervisory staff from the mining industry; the papers were generally descriptive in nature, essentially outlining the procedures and techniques being used at various mines for the disposal of tailings. Within the last 5 to 7 years, however, a host of papers have been presented by geotechnical engineers, outlining design requirements for safe tailings dams based on considerations of basic principles of soil mechanics and earth dam engineering. These papers have generally tended to be qualitative in nature.

In summary, interest in the safe design and construction of tailings dams has been steadily growing in the last decade or so, as evidenced by the increase in technical papers on the subject culminating in the First International Tailings Symposium held in Tucson, Arizona, in 1972.

1.8 Scope, Objectives and Organization of the Thesis

Tailings dams may be constructed of waste rock from the mining operations, borrow fill from other sources, or predominantly of sands recovered from tailings. It is thought that where waste rock or borrow fill is used as construction materials, the tailings embankments can be adequately designed by procedures commonly followed in conventional rock and earth fill dams. Furthermore, requirements and considerations for site investigations, and assessment and preparation of foundation materials are essentially the same as those for conventional water storage dams. Therefore, only the embankment itself, when it is to be constructed primarily of tailings sand, is considered in this study. The remarks made in this thesis, can also be taken to apply when borrow materials are used to augment the available sand supply.

The objectives of this study are to investigate and synthesize requirements for design and materials handling for construction of tailings dams. In order to achieve the objectives of the study a detailed and systematic investigation is first undertaken to determine physical properties of tailings materials and obtain much needed performance data on the more important factors affecting the design of tailings dams. Contents of the various chapters of the thesis are summarized below.

Chapter II presents an outline of the basic principles of soil mechanics and earth dam engineering which are considered to constitute the present state-of-the-art of design and construction of tailings dams. A brief outline of the critical review of the present state-of-the-art and possible areas for intended research is also included in this chapter.

The details and test results from a systematic laboratory testing program undertaken as part of this study are presented in Chapter III. Various tailings materials from several typical mining operations are included in this testing program. Where necessary for comparison purposes, standard Ottawa sand and sand from the Fraser River in British Columbia are also included.

Field investigations are undertaken to perform necessary testing and collect performance data on the more important factors affecting the design of tailings dams. All pertinent details are presented in Chapter IV.

Chapter V outlines a new method of computing seepage flow through tailings dams. All other related details are also included.

A review of the possible earthquake effects on the stability of tailings dams to be constructed in seismic areas

is presented in Chapter VI.

Chapter VII deals with the synthesis of design criteria relevant to the selection of construction materials and materials handling. A few suggestions for improvements in construction techniques are also discussed.

Chapter VIII presents a summary of conclusions drawn from this investigation and a few suggestions for future research.

TABLE 1.1 List of Failures of Tailings Dams

Name of Dam	Construction Started	When Failed	Height of Failure (ft.)	Slope Angles	Cause of Failure	Mechanism of Failure	Method of Construction	Reference	Remarks
Barahona, Chile (copper tailings)	1920	1928	200	40°-45°	Earthquake	Liquefaction of slimes, and inward slide of embankment.	Upstream method using separator cones to build exterior dyke.	Dobry and Alvarez (1967)	Embankment 50 ft high over pond level. 4 million tailings released. 54 people killed.
Chemical Waste Spoil Bank Louisville, Kentucky	-	1963	100	-	Heavy rains		Similar to upstream method.	Smith (1969)	For comments on chemical waste embankments, see Casagrande (1950).
Dam No. 7 Mulfulira Copper Mines Ltd. Zambia	1941	1952	68		Heavy rains	Seepage near toe and local sloughing first	Upstream construction	Finn (1965)	Dam totally failed discharging bulk of the stored tailings.
Zinc ore tailings deposit						Failure through soft foundation soils	Upstream construction	Salas (1969)	Dam built on previously deposited slimes.
Iron ore tailings deposit			40		Heavy rains	Failure through foundation soils	Dam built of borrow material on old fill.	Salas (1969)	
Buffalo Creek Coal Waste Dam West Virginia		1972	80	35°	Rain storm	Massive slide in the downstream section	The dams were built by end dumping of coal refuse from trucks and grading with bulldozers.	Wahler and Associates (1973)	Coal refuse dam failed. The consequences of the resulting flooding were deaths of 118 persons and 7 reported missing. 500 homes destroyed.
New Dam El Cobre, Chile (copper tailings)	Dec. 1963	Mar. 1965	60	15°	Earthquake	Increased pressure from liquefied slimes opened a gap in a corner of the triangle.	Downstream construction cyclones used to separate sand.	Dobry and Alvarez (1967)	A triangle in plan area. Dam completely destroyed. Liquefied slimes flowed 7 miles killing 200 people.
Old Dam El Cobre, Chile (copper tailings)	1930	1965	100	35°-40°	Earthquake	Liquefaction of tailings.	Upstream construction.	Dobry and Alvarez (1967)	A square in plan area.

TABLE 1.1 List of Failures of Tailings Dams (cont'd)

Name of Dam	Construction Started	When Failed	Height of Failure (ft.)	Slope Angles	Cause of Failure	Mechanism of Failure	Method of Construction	Reference	Remarks
Hierro Viejo Chile (copper tailings)		1965	15	35^o-40^o	Earthquake	Liquefaction of tailings.	Hydraulic filling and intermittent drying.	Dobry and Alvarez (1967)	A triangle in plan area.
Los Maquis, Chile (copper tailings)	1937	1965			Earthquake	Liquefaction of tailings.	Upstream construction	Dobry and Alvarez (1967)	Liquefied tailings flowed 2 miles towards a river.
La Patagna, Chile (copper tailings)		1965		35^o	Earthquake	Liquefaction of tailings.	Upstream construction	Dobry and Alvarez (1967)	Liquefied tailings flowed 2 miles but no damage was done.
Cerro Negro, Chile (copper tailings)	1961	1965			Earthquake	Liquefaction of tailings.	Upstream construction	Dobry and Alvarez (1967)	Liquefied tailings flowed 2 miles but no damage was done.
El Cerrado, Chile (copper tailings)	1955	1965	80	35^o	Earthquake	Slides in the corners.	Upstream construction	Dobry and Alvarez (1967)	
Bellavista, Chile (copper tailings)		1965	50	30^o-35^o	Earthquake	Liquefaction of tailings.	Upstream construction	Dobry and Alvarez (1967)	Dam failed completely.
Sauce, Chile		1965	20	30^o	Earthquake	Fractured badly at corners.	Upstream construction	Dobry and Alvarez (1967)	Dam did not fail completely.
Ramayana, Chile		1965	15	33^o	Earthquake		Upstream construction	Dobry and Alvarez (1967)	Dam partly destroyed.
Cerro Blanco de Polpaico (non-mettalic)		1965	25		Earthquake	Vertical fractures.	Used riprap on downstream slope.	Dobry and Alvarez (1967)	Dam did not fail completely.

NOTE: Smith (1967) reports several other failures in Spain and Hoare (1972) in Canada but sufficient details are not available for inclusion in the above table.

CHAPTER II

CURRENT PRACTICE OF DESIGN AND CONSTRUCTION OF TAILINGS DAMS

2.1 Introduction

Traditionally, tailings dams have been constructed by the mining companies generally using in-house empirical expertise based on many years of experience and observation but with little or no consideration for the appropriate engineering principles and hence, with frequent failures in certain cases. For example, Gordon (1966) reports a case where one of the larger mining companies experienced seventeen failures over a period of seven years at a single mining property.

In the last decade, in view of these failures, the growing awareness of the public for the protection of the environment and the need for pollution control, the design of tailings dams has come under critical scrutiny by the governmental regulatory agencies. Consequently, stringent regulations have been established or are forth-coming shortly to control the design and construction of tailings dams. At the present time, the general consensus is that the tailings embankments must be designed to meet

standards comparable to those which are considered necessary for the safe design of conventional water storage dams.

A detailed consideration of the many factors affecting the design of tailings dams is essentially beyond the scope of this study. Thus, only a brief description of the basic concepts relevant to soil mechanics and materials handling which are considered to represent the present state-of-the-art of design and construction of tailings embankments is presented in this chapter under the following headings:

i) different construction methods,

ii) design criteria, and

iii) thoughts on inherent characteristics of tailings embankments constructed by the different methods.

A more complete description of all factors relevant to the design and construction of tailings dams has been adequately covered in the "Tentative Design Guide for Mine Waste Embankments in Canada" by the Department of Energy, Mines and Resources (1972).

A critical review of the present state-of-the-art and possible areas for intended research is also included in this chapter.

2.2 Different Construction Methods

In all tailings embankments, initially, a relatively low starter dam is constructed of borrow material using methods and design criteria for conventional water storage dams. The "starter dam" provides a reservoir of sufficient size for the storage of water for mill start-up and tailings disposal during the initial stages of the milling operation. As the pond level rises and approaches the crest level of the starter dam further suitable material is added as required to raise the height of the embankment.

The design of a tailings dam is greatly influenced by the method of construction used in continuously raising the height of the embankment to maintain a crest level higher than the ever rising pond level due to the disposal of tailings in the reservoir behind the dam. When an embankment is constructed predominantly of sand recovered from the tailings, there are three basic methods of construction which are commonly employed. These construction methods can be described as:

- the upstream method,
- the downstream method, and
- the centreline method.

2.2.1 The Upstream Method of Construction

In this method, the crest of the dam is progressively shifted in an upstream direction as the height of the

dam increases so that the starter dam forms the downstream toe of the ultimate embankment. This is the oldest method of constructing tailings dams, and is a natural development from the procedure of disposing of tailings as cheaply as possible. Total tailings are discharged by spigotting in an upstream direction off the top of the starter dam. The height of the embankment is raised as required by placing dykes as shown in Figure 2.1. Except for the initial starter dam, all subsequent dykes rest in part on the dyke below and in part on the surface of the previously spigotted tailings. These dykes are constructed of materials obtained by one of the following methods:

i) generally by dragging up sandy material from the tailings beach which forms in front of the spigots,

ii) sometimes by borrowing material from the mine spoil or other suitable source, or

iii) by using cyclones to separate sand from the tailings to expedite construction of the dykes.

Another method of raising the height of the dam, often followed in the past and still followed at a few sites, involves using vertical timber forms to raise the dam in 2 to 3 foot increments. Figure 2.2 is an illustration of a dam being built by this method of construction. The tailings dam at Craigmont Mines to be discussed in a later chapter is being constructed by this technique.

2.2.2 The Downstream Method of Construction

Construction is carried out in this method by placing the coarser fraction of tailings (sand) on the downstream side of the starter dam so that the crest of the embankment is progressively shifted in a downstream direction and the starter dam forms the upstream toe of the final dam. Although Jigins (1957) has reported a tailings dam in Chile being constructed by the downstream method in 1929, this method of constructing tailings dams is generally considered to be a relatively new development. Figure 2.3 illustrates a tailings embankment being constructed by the downstream technique.

2.2.3 The Centreline Method of Construction

This type of construction is, essentially, a variation of the downstream method where the crest of the embankment is not shifted in the downstream direction but raised vertically upwards above the crest of the starter dam. The centreline technique is illustrated in Figure 2.4.

2.3 Design Criteria

Although there are numerous minor and detailed considerations involved in the design of a tailings dam, the most critical items are those related to the seepage of water and the stability of the structure under both static and earthquake loading conditions. A brief discussion of these critical factors relevant to the design of

tailings dams follows.

2.3.1 Seepage Control

2.3.1.1 Overview

As in the case of a conventional water storage dam, there are two requirements for seepage control for a tailings dam that must be considered - one that relates to the quantity of water seeping through the dam and the other that relates to its effects on the stability of the structure due to high pore pressures and/or high seepage pressures and resulting erosion. Requirements for water supply and pollution control may necessitate the use of special design features such as impervious liners, impervious cores, cut-offs, etc., to minimize seepage through the dam. But usually, it is the second requirement which is more critical and warrants careful consideration in the design of tailings dams. With regard to this aspect of seepage control, there are two basic conditions that must be met in the design - one, to improve stability, the phreatic surface must be maintained well within the embankment and the other, to minimize erosion and a possible piping failure, water must not be allowed to seep through the face of a sand slope. Further comments on the effect of these two design requirements on the stability of the tailings dams follow.

2.3.1.2 Effect of Phreatic Surface on Stability

The position of the phreatic surface within an

embankment has a marked influence on its stability. As an example, let us consider the case of an infinite slope comprised of cohesionless sand - first with and then without seepage. When there is seepage through the slope so that the phreatic surface or water table coincides with the surface of the slope, the steepest angle at which this slope will be stable, regardless of the height of the slope, can be computed from

$$\tan \alpha = \frac{\gamma_t - \gamma_w}{\gamma_t} \times \tan \phi' \qquad \text{.....(2.1)}$$

where α is the slope angle,

γ_t is the total unit weight of the material in the slope,

γ_w is the unit weight of water, and

ϕ' is the angle of shearing resistance of sand in terms of effective stress ($c'=0$).

Assuming a value of 120 pounds per cubic foot for γ_t and of 35 degrees for ϕ', we get 18-1/2 degrees for the slope angle (3 horizontal to 1 vertical). Whereas, when there is no seepage through the slope, the steepest angle for the slope is equal to the angle of shearing resistance of the sand which is 35 degrees (1.4 horizontal to 1 vertical) - more than twice as steep as that for the with-seepage case. The situation in a tailings dam usually is between these two extremes. Nevertheless, it is indicated that the lowering of phreatic surface increases the stability of the embank-

ment, thereby permitting steeper downstream slopes and resulting in a reduced quantity of fill required for construction.

In the case of tailings dams, in addition to the stability considerations under static loading conditions as discussed above, the lowering of the phreatic surface also increases stability against failure by liquefaction under earthquake or dynamic loading. This point is discussed in more detail in a later section of this chapter.

2.3.1.3 Effect of Seepage Through Downstream Slope on Stability

The force imparted to the soil grains by seepage of water may be computed from

$$A = i\,\gamma_w \qquad \text{....(2.2)}$$

where A is seepage force in lb/cu ft

i is hydraulic gradient, and

γ_w is unit weight of water in lb/cu ft.

Thus, if the seeping water has a large exit gradient when leaving the downstream slope, a potentially unstable situation may arise and a piping failure may result. Even if the exit gradient is small, water seeping through a sand slope can cause localized erosion and hence, instability near the toe of the slope and ultimately the entire slope if left to run its course. To prevent piping failure or erosion due to the seeping water, protecting filters are commonly employed. The portion of the slope

where seepage occurs, is, nevertheless, subject to considerations discussed in the previous section.

In a properly designed seepage control, consideration also must be given to eliminate the detrimental effects of frost penetration. In areas where seepage is occurring, frost penetration may result in impeded drainage and thus a higher phreatic surface and sloughing of the slope on subsequent thawing.

Figures 2.5 to 2.7 illustrate various methods of seepage control employed in the case of conventional water storage dams. Similar techniques, properly modified, can provide adequate seepage control in the case of tailings dams.

2.3.2 Stability Considerations

A list of minimum safety factors suggested in the "Tentative Design Guide for Mine Waste Embankments in Canada" by the Department of Energy, Mines and Resources (1972) is given in Table 2.1. It is assumed that an appropriate stability analysis has located the critical failure surface and that the parameters used in the analysis are known with reasonable certainty to be representative of the actual conditions that will exist in the embankment.

Where the ratio of residual to peak shear strengths is 0.9 or greater, the embankment design can be based on the peak strength values using the appropriate factors of safety. Where the number of field and laboratory test results on either the embankment fill or the foundation soils is small, or where the scatter of test results within individual strata or zones is large, conservative values of strength and pore water pressures should be selected for the design, or, alternatively, an increased factor of safety should be used.

In addition to the stability considerations summarized in Table 2.1, in seismic areas, tailings dams must also be designed to have an adequate factor of safety against failure by liquefaction. Loose and saturated fine to medium sands, similar to those used to construct tailings dams, are susceptible to liquefaction when subjected to cyclic shear stresses and strains produced by earthquake shocks. It is generally accepted, now, that for any given saturated sand, the danger of liquefaction as a result of cyclic loading is governed by the following factors (Seed and Lee, 1966):

i) the density of the sand - the lower the density, the more easily liquefaction will occur;

ii) the confining pressure acting on the sand - the lower the confining pressure, the more easily liquefaction will occur;

iii) the magnitude of the cyclic stresses or strains - the larger the stress or strain the greater the probability that liquefaction will occur;

iv) the number of stress cycles to which the sand is subjected - the greater the number of stress cycles, the more easily liquefaction will occur.

It is noteworthy that the embankment design cannot influence the magnitude of the cyclic stresses or number of cycles to which the dam may be subjected. Through proper design and appropriate construction procedures, however, the other two factors might be controlled.

The general consensus among the engineers involved in designing tailings dams is that in situ relative densities of 60 percent or larger are required to ensure a reasonable safety against failure by liquefaction. See for example, Brawner (1972); Casagrande, L. (1971); Department of Energy, Mines, and Resources (1972); and Klohn (1972B).

Another rather interesting approach is that non-saturated sands are not likely to liquefy. Hence, if the phreatic surface can be maintained at a position well below the surface of the embankment, those materials located above the phreatic surface will not be susceptible to liquefaction. Lowering the water levels within the embankment also

increases the effective confining pressures acting on the saturated materials below the phreatic surface and thereby reduces their susceptibility to liquefaction.

The design requirements to prevent failure by liquefaction have been summarized in the "Tentative Design Guide for Mine Waste Embankments in Canada" by the Department of Energy, Mines and Resources (1972) as follows:

> "....., providing the materials comprising the embankment have a relative density of 60 per cent or greater, and/or providing the phreatic surface is maintained at a position well below the surface of the embankment, the embankment itself will have a reasonable degree of safety against failure by liquefaction. Liquefaction of the impounded tailings adjacent to the upstream face of the embankment may nevertheless occur. The mass of the embankment should be sufficient to provide the required factor of safety against horizontal displacement along its base, when the assumption is made that the shear strength of the tailings adjacent to the upstream face is reduced to zero."

However, some engineers argue that compacting of the sand fill to relative densities cited above is the only positive way to ensure safety against failure by liquefaction (Brawner and Campbell, 1973).

2.4 Thoughts on Inherent Characteristics of Tailings Embankments Constructed by Different Methods

2.4.1 Overview

As described previously in Section 2.2 of this chapter, there are basically two different methods of constructing tailings dams - the upstream method and the

downstream method (with the centreline method as its special case). Almost all of the existing tailings embankments have been constructed using the upstream technique. Hoare (1972) reports that

> "A review of the national survey on tailings dams indicates that 95% of the hydraulic dams in Canada are constructed by spigotted methods."

In view of the many failures that have been associated with tailings dams, the design of these structures has come under critical review in the last few years. The downstream method of constructing tailings dams appears to be essentially the result of these recent reviews. At the present time, the general consensus in the engineering profession is that for a given height and downstream slope, a dam constructed by the upstream method has a lower factor of safety than that of a dam built by the downstream or centreline methods and that the dam built by the upstream method is in particular danger of failure by liquefaction in the case of an earthquake. Some engineers even go as far as to state that all but very minor tailings dams should be built by the downstream or centreline methods of construction (Klohn, 1972B). A more detailed description of current thoughts on the inherent characteristics of the tailings dams constructed by the different methods follows.

2.4.2 Tailings Dams Constructed by the Upstream Method

2.4.2.1 General Background

Although no definite figures are available, it is generally considered that the chief advantage of the upstream method of constructing tailings dams is its simplicity and resulting low cost of construction. Except for construction of the relatively small exterior retaining dykes, all material is discharged hydraulically in an upstream direction so that excess tailings along with the water flow into the pond.

As the upstream method appears to have gained popularity with the mining operators mainly because of the low cost and simplicity of construction, by and large little or no attention has been paid to even the most basic principles applicable to the design of earth-fill dams. Thus, many dams built by this method have failed, some with great loss of life and property as listed in Table 1.1 in the previous chapter.

2.4.2.2 Construction and Materials Handling

With the upstream method, an initial starter dam is constructed at the downstream toe of the final dam. The crest of the dam is raised by placing subsequent dykes located on the upstream side of the starter dam. All tailings are spigotted in an upstream direction from the top of the dam so that a tailings beach is usually formed

between the spigots and the pool in the reservoir. A typical layout for a dam being built by this method is illustrated in Figure 2.1.

In the upstream method, tailings are subjected only to gravity separation on the beach - the coarse sand particles drop out closest to the point of discharge and the finest particle sizes (slimes) along with the excess water flow into the pond. It is generally believed that once the material enters the water in the pool it sediments more or less as a homogeneous slurry into a consistency of very soft silt. As the length of the beach can vary according to the time of the year, with the shortest length occurring during the spring runoff into the pond, the beach deposit may be underlain by thin layers of soft slimes (unsorted fine tailings that sediment initially in water into a consistency of very soft silt) deposited during these periods of high water levels in the pond. Also careless spigotting, for example, infrequent change in areas being spigotted, can cause back-ponding in the neighboring beach areas with the same results.

The coarse material (sand) from the beach is generally used to build the subsequent dykes required to raise the height of the dam as previously discussed in Section 2.2.1.

To expedite construction of the dykes, on occasion, hydrocyclones are employed to separate sand from the tailings but, by and large, the tailings are discharged in an upstream direction in a manner similar to that used in the "all-spigot" operation described above.

Generally, the material in the dykes or the beach is not subjected to any mechanical compaction other than that which may be obtained by normal operation of equipment used for construction of the dykes.

2.4.2.3 Stability Considerations

As the height of a dam constructed by the upstream method increases, each successive dyke moves farther upstream. Consequently, the dykes form only the thin exterior shell of the retaining embankment. Thus, with the increasing height of the dam, the stability of the downstream slope is dependent, to an ever increasing degree, on the shear strength of the tailings deposited upstream of the dykes. Furthermore as the height increases (depending on the length of the beach, of course) the upper portions of the dam may overlie soft slimes previously deposited near the base of the embankment or in thin layers within the beach sand.

Qualitative stability considerations will tend to indicate that there is perhaps only a limited height to which tailings dams can be constructed under these circum-

stances without the danger of a failure in the downstream direction through these underlying layers of low strength slimes. Furthermore, tailings dams constructed by this method are generally considered to be particularly susceptible to failure by liquefaction at any height (Casagrande and MacIver, 1971). The present thinking on the suitability of this method for constructing tailings dams has been summarized by Klohn (1972B) as follows,

> "Upstream dam building is unsuitable for areas where the dam must be designed to resist earthquake shocks. Even in non-seismic areas this method of construction is generally unsuitable for all but very minor tailings dams."

2.4.3 Tailings Dams Constructed by The Downstream or Centreline Method

2.4.3.1 Overview

At the present time, it is thought that the downstream and centreline methods of construction are far superior to the more traditional upstream construction and that by these methods tailings dams can be constructed to any degree of competency (including resistance to earthquake forces). Some of the desirable features of constructing tailings dams by these methods are

i) none of the embankment is built on top of the previously deposited slimes,

ii) placement and compaction control can be exercised as required over the fill operation, and

iii) underdrains can be installed for seepage

control as required.

The major disadvantage of the downstream method of construction is the large volume of sand required to raise the dam. This problem is somewhat reduced by using the centreline method which requires a smaller volume of sand fill to raise the crest of the dam to a given height. In the critical early stages of the operation, however, still it may not be possible to produce sufficient volumes of sand to maintain the crest of the dam above the fast rising pond levels. If this is the case, then either a higher starter dam is required or the sand supply must be augmented with borrow material. Both of these procedures add to the initial cost of the facility.

Another disadvantage of the downstream construction method is that the downstream slope is always changing during construction and is therefore constantly exposed to erosion by wind and rain water. No erosion protection can be applied to the slope until after the dam is fully completed. A further discussion of some of the more salient features which control the viability of the downstream methods of constructing tailings dams follows.

2.4.3.2. Methods of Separating Sand from Tailings

All methods of constructing tailings dams which utilize tailings as the primary construction material require

some degree of separation of the sand from the tailings. Whereas the conventional upstream method utilizes spigots to obtain gravity separation of sand on the beach which forms in front of the spigots, the downstream method normally uses hydrocyclones. The cyclones permit deposition of sand for dam building downstream of the starter dam and disposal of slimes (finer portion of tailings) directly into the storage pond. For grinds that contain a high percentage of fines and/or clay minerals, double cycloning is considered necessary to produce suitably clean sand for dam construction, Klohn (1972A, 1972B). Cyclones may be located in clusters on the dam abutments, with sand piped to the dam, or as individual units on the dam itself.

An alternate process of obtaining sand for the centreline method of dam building uses conventional upstream spigotting to obtain gravity separation on an upstream beach and then, by means of mechanical equipment, moves the sand to the downstream slope as illustrated in Figure 2.8. This method, however, is not likely to be practicable where large volumes of sand are required (Casagrande and MacIver, 1971).

Hydraulic sluicing methods are sometimes used to obtain separation of sand from slimes. These are a refined form of gravity separation. The tailings are deposited at one end of a long and gently sloping channel

along the crest of the dam, with the sand settling out along the base of the channel and the slimes remaining in suspension to be discharged at the lower end of the channel into the storage pond. By means of mechanical equipment the sand is then moved to the downstream slope. Figure 2.9 illustrates this method. A variation of the channel sluicing method is to use pipe sluices consisting of half-pipe sections incorporating baffles and openings in the bottom of the pipe. Coarser sands are retained by the baffles and drop through the openings in the pipe onto the embankment; the finer slimes flow on to the storage pond. Hydraulic sluicing, however, is not likely to be suitable where large volumes of sand are required unless tailings contain a large percentage of sand sizes.

2.4.3.3 Characteristics of Sand

One of the most critical factors affecting the design and construction of tailings dams by the more desirable downstream methods is the characteristics of the sand incorporated in the embankment. The engineers involved in designing tailings dams believe that the sand must be free draining. The amount of fines (or slimes) in the sand is considered to control the quality of the sand. The following statement taken from the "Tentative Design Guide for Mine Waste Embankments in Canada" by the Department of Energy, Mines and Resources (1972) seems to summarize the present thinking on the subject.

> "If the sands deposited on the embankment have a relatively high content of clay-like 'slimes', they will not only be slow in draining to optimum water content, but they will be difficult to handle. Coming from the cyclones with a high water content and a high slimes content, they will flow on very flat slopes, even to the extent of spreading beyond the design boundaries of the embankment. In contrast, if the slimes content is low, water will drain rapidly from the sands, limiting the distance they flow on the embankment and allowing spreading and compaction by mechanical equipment a short time after their deposition."

A review of the characteristics of sand being used at some of the new tailings dams under construction reported in the literature, as given in Table 2.2. appears to indicate that the slimes content is restricted to less than about 12%.

2.4.3.4 Methods of Sand Placement and Compaction

In the downstream construction, various techniques of sand placement and compaction are presently being tried, ranging from hydraulic filling, (Klohn and Maartman, 1973) to systematic placement and compaction in thin layers by mechanical equipment, (Brawner, 1972).

2.4.3.5 Seepage Control

In the downstream methods of construction, the cycloned sand is deposited on the dam in a downstream direction and the slimes are discharged into the storage pond.

In addition to a system of underdrains and/or a toe drain, an impervious seal against the upstream face of the dam is usually considered necessary for seepage control to maintain a minimum flow through the dam and a low phreatic surface within the sand embankment. Various techniques of employing an impervious seal can be used. Brawner (1972), and Hoare and Hill (1970) describe impervious seals of borrow material placed mechanically against the upstream slope of the dam. Klohn and Maartman (1973) describe how a low-permeability slimes beach is formed against the upstream face of the dam at Brenda Mines by conventional spigotting of the slimes in an upstream direction from along the crest of the dam.

2.5 A Critical Review of the Present State-of-the-Art And Possible Areas for Intended Research

2.5.1 Historical Background

As discussed previously in Section 2.1, traditionally tailings dams have been constructed by mining operators using in-house staff with a minimum involvement from the geotechnical profession. In the last decade or so, however, with the ever increasing through-put capacity of the low grade open pit mines requiring some high tailings dams and in view of the many failures of tailings dams, there has been a trend towards stricter regulations whereby geotechnical engineers are required to design and make recommendations for the construction of tailings dams. Consequently, in the last few years the geotechnical profession

has been hard pressed to produce suitable guidelines for constructing safe tailings dams. The geotechnical profession has met the challenge rather expeditiously and put together appropriate design criteria and guidelines as indicated by the many references cited throughout this chapter. All the engineers responsible for these efforts are to be complimented for doing a commendable job under demanding circumstances and without a precedent to follow.

It must be recognized that many of the design criteria which constitute the present state-of-the-art have been established without detailed knowledge of the physical properties of tailings and benefit of performance data which has been late in coming. Hence, at least some of these criteria must be considered, at best, interim in nature. With this in mind, we must explore possible areas for further research so that safe tailings dams may be constructed without the possible wastefulness of over-design.

2.5.2 Upstream Construction

Except for Kealy and Soderberg (1969), Kealy (1973), and Blight (1969) all previous investigators appear to have concentrated on finding reasons why tailings dams constructed by the upstream method (as now being employed at many mines) are subject to failure. Consequently, sufficient attention has not been paid to delineate

operating conditions under which the upstream method (as now used) or some variation of it may be used to construct safe tailings dams.

2.5.3 Downstream Construction

2.5.3.1 Seepage Control

The basic technology and experience available from the field of conventional water storage dams has been used extensively in arriving at the design requirements for tailings dams (Klohn, 1972B). It is noted, however, that there are at least two basic differences in these two types of dams. First, unlike the conventional water storage dam, a tailings dam is constructed slowly by the mining operator and the construction usually continues throughout the operating life of the mine. Second, the material stored behind a tailings dam consists of a mixture of tailings solids and water.

These differences are well recognized in stability analyses when increased hydrostatic pressure is considered acting against the upstream slope to account for the combined unit weight of water and tailings solids. In assessing requirements for seepage control, however, the effect of these differences is perhaps much more subtle and requires further examination.

When water rests directly against the slope of

an embankment, to estimate the rate of seepage, a flow net construction is usually employed, assuming a steady state seepage condition; and the location of the phreatic surface can be estimated by the methods described by Casagrande (1937). In the case of the tailings dam, however, it is a mixture of soil solids and water that rests against the dam with a possible shallow depth of free water at the surface. Furthermore, where rate of slimes build-up in the pond is rather rapid, the tailings against the dam may be in an under-consolidated state with excess pore pressures. Under these circumstances, a flow net construction as presently employed in the tailings dams is anything but appropriate and not likely to produce a realistic estimate of the rate of seepage through the dam. Need for the development of a more appropriate analysis, therefore becomes obvious.

As previously discussed in Section 2.4.3.5, for seepage control an impervious seal is frequently used against the upstream face of a tailings dam. This approach would have been considered quite appropriate in the case of a conventional water storage dam with a homogeneous section. For a tailings dam, however, where tailings and water rest against the dam with only a possible shallow depth of free water at the surface, need for this type of impervious seal becomes questionable.

2.5.3.2 Sand Yield

Perhaps, the most critical factor affecting the design and construction of tailings dams built by the downstream method using tailings sand as the primary construction material is the yield of suitable sand from the tailings. Variations in sand yield have a dual effect - a decrease in sand yield results in a decrease in the rate at which the crest of the dam can rise and an increase in the rate of rise of the pond level. The sand yield is dependent upon the coarseness of the grind, the efficiency of the separating technique (hydrocyclones) and the amount of slimes that can be accepted in the sand. But coarseness of the grind is generally governed by the milling requirements to liberate valuable minerals and hence, the sand yield is dependent upon the other two variables only. A detailed further investigation into these two variables is, therefore, of paramount importance.

2.5.4 Stability Under Earthquake Loading

In areas subject to earthquake activity, failure by liquefaction poses the most serious potential problem. Under earthquake loading, sands of gradations common to tailings dams are susceptible to liquefaction when in a loose and saturated state.

A brief review of case histories where liquefaction has been reported to have taken place appears to

indicate that liquefaction occurs only when materials are loose and saturated or at least nearly saturated. It is therefore logical to say that main protections against liquefaction are drainage and densification. It can be further said that these two requirements may be mutually exclusive, at least under suitable circumstances. Although Klohn and Maartman (1973) have done some work in this area, it is felt that a thorough review of the available information on the case histories of liquefaction is certainly warranted.

Seed and Lee (1966) have shown that confining pressure has a marked influence on the liquefaction potential of a soil. All other things being equal, an increase in confining pressure results in a decrease in liquefaction potential. It appears that this principle has been used to reduce liquefaction potential in the case of nuclear power plants (Burke, 1973B). In the case of tailings dams, however, although the principle is recognized as being beneficial (Klohn, 1972B), it has not been properly evaluated. Further research in this area is, therefore, likely to be of some value.

2.5.5 Physical Properties of Tailings

Generally speaking, there is lack of information on physical properties of tailings. The need for a systematic study of physical properties of the materials has

been adequately summarized by E. E. Osborn (1973), Director of U.S. Bureau of Mines, when he said,

> "......Some facets of earth dam technology, soil mechanics, rock mechanics and engineering geology can provide some of the basic knowledge needed to bring the mine waste disposal problem under control. However, physical properties of most mine waste materials have never been thoroughly studied. Consequently, their general and engineering characteristics are simply not known, even to the degree necessary for rapid, accurate assessment of the suitability of current disposal systems - much less for the design and construction of new and better systems. Lamentably, a time-consuming program of field and laboratory investigation must first be carried out if there is to be proper safety evaluation of the materials involved in mine waste embankments."

2.5.6 Possible New Methods of Construction

At the present time, it is generally believed that the upstream method is likely to be unsuitable for at least the larger tailings dams of the future; and hence, the downstream method must be used. A review of the previous sections of this chapter indicates that both of these methods have some good points and some inherent weaknesses. Neither method is faultless. Further work is, therefore, in order to investigate alternative methods or perhaps, some variations of the above that reduce some of the undesirable features.

TABLE 2.1 MINIMUM SUGGESTED DESIGN SAFETY FACTORS
(After the Department of Energy, Mines and Resources, 1972)

	Case 1*	Case 11**
Design based on peak shear strength parameters	1.5	1.3
Design based on residual shear strength parameters	1.3	1.2
Analysis that include the predicted 100-year return period accelerations applied to the potential failure mass	1.2	1.1
For horizontal sliding on base of embankments retaining tailings in seismic areas assuming shear strength of tailings reduced to zero	1.3	1.3

* Case 1 - Where it is anticipated that persons or property would be endangered by a failure.

** Case 11 - Where it is anticipated that persons or property would not be endangered by a failure.

TABLE 2.2 CHARACTERISTICS OF SANDS BEING USED AT NEW DAMS

Percent Passing #200 Sieve	Name of Dam or Location	Reference
3*	Brenda	Klohn and Maartman (1973)
7*	White Pine	Girucky (1973)
10	Gibraltar	Klohn and Maartman (1973)
12**	Marindugue Island Philippines	Brawner (1972)
12	Typical cycloned sand	Burke (1973A)

* - Twice cycloned as single cycloning did not produce suitable sand.

** - Estimated from given D_{10} = .07 mm

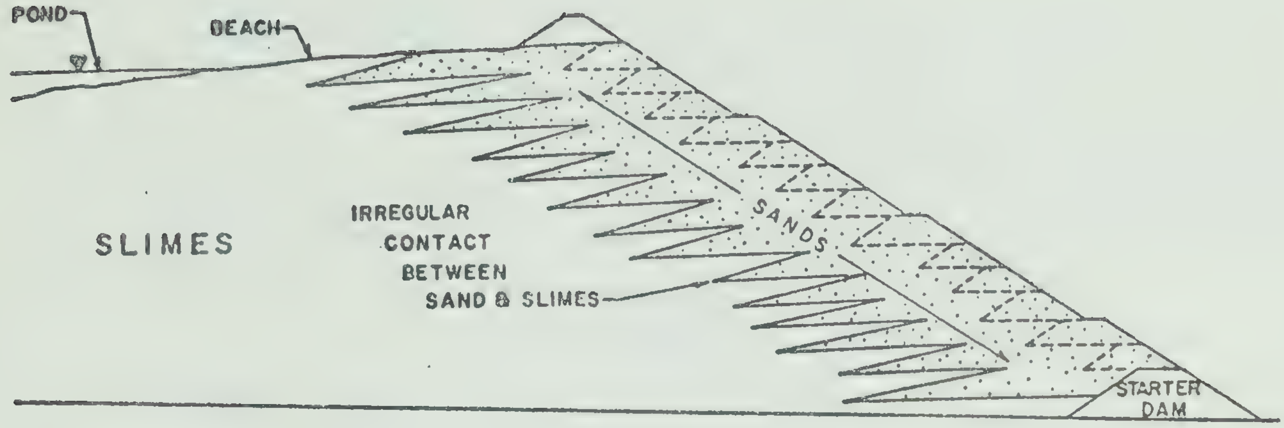


Figure 2.1 Upstream Method of Construction

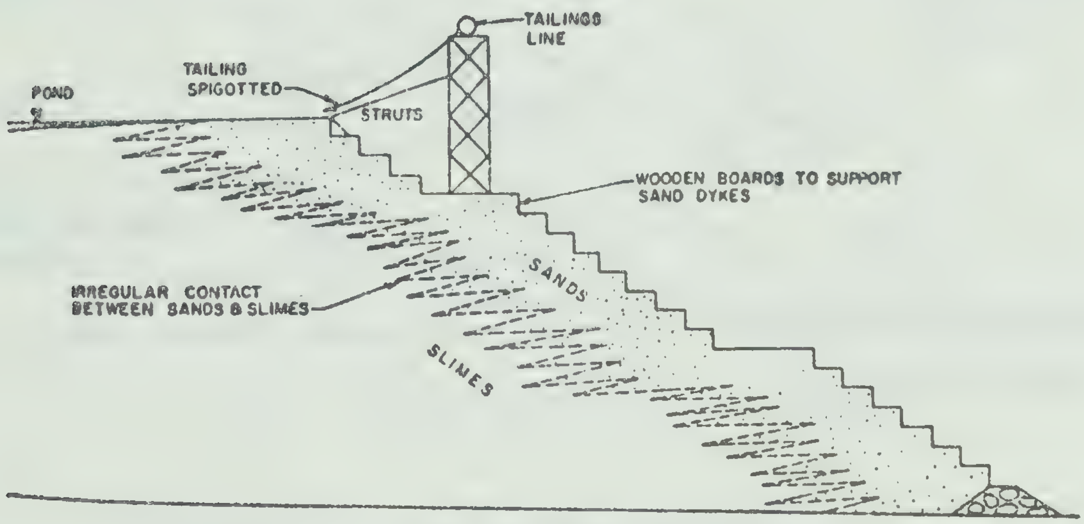


Figure 2.2 Upstream Construction Using Timber Forms

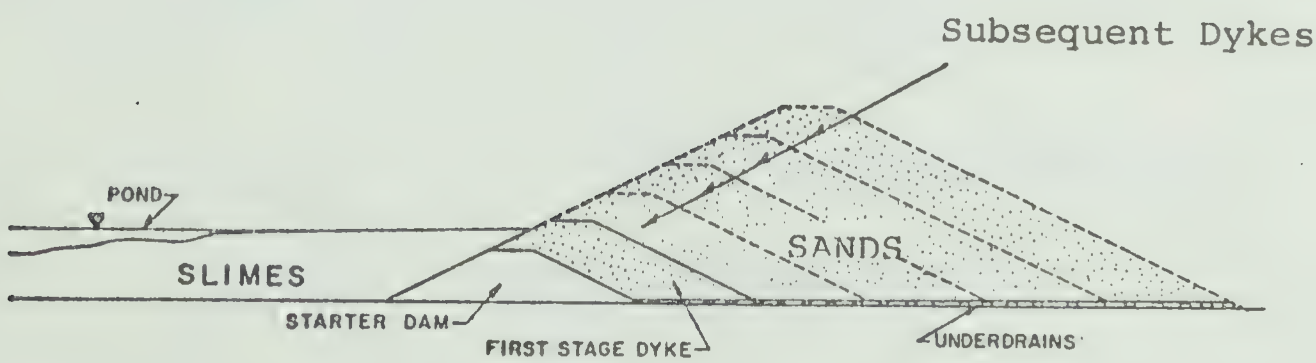


Figure 2.3 Downstream Method of Construction

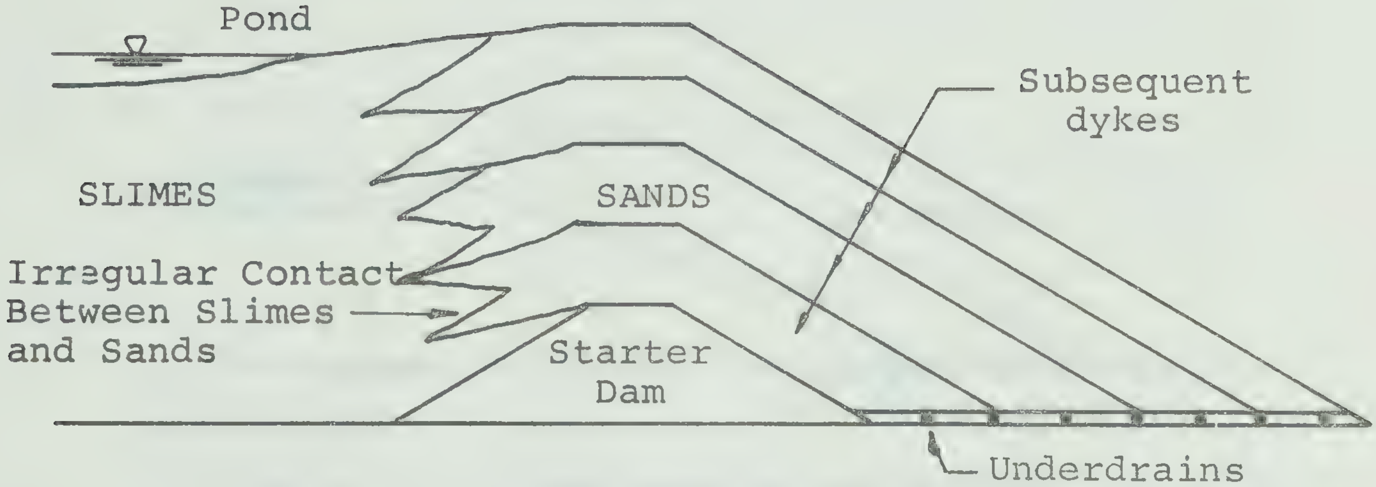


Figure 2.4 Centreline Method of Construction

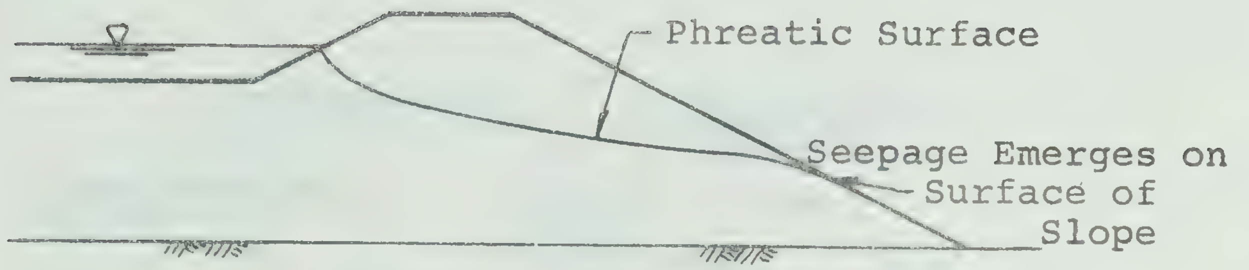


Figure 2.5 Homogeneous Dam
(No Drainage Facility Provided)

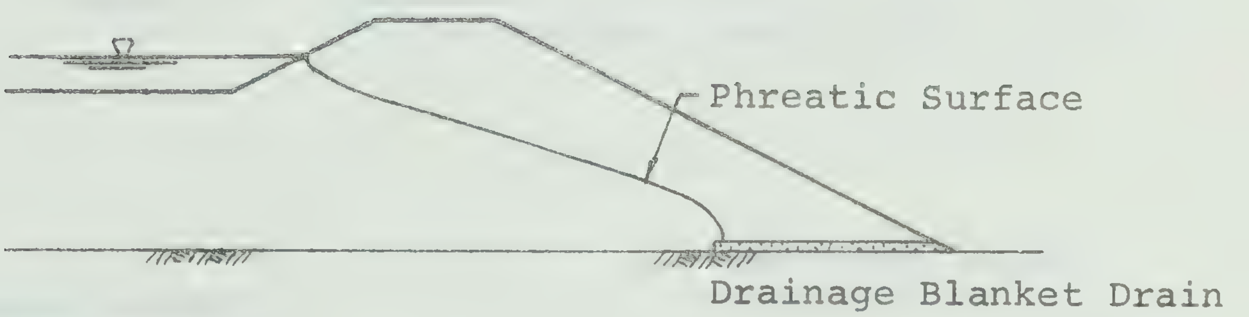


Figure 2.6 Homogeneous Dam With
Underdrains

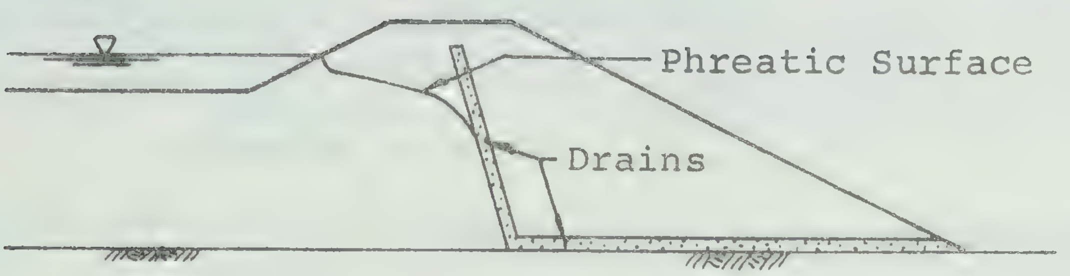
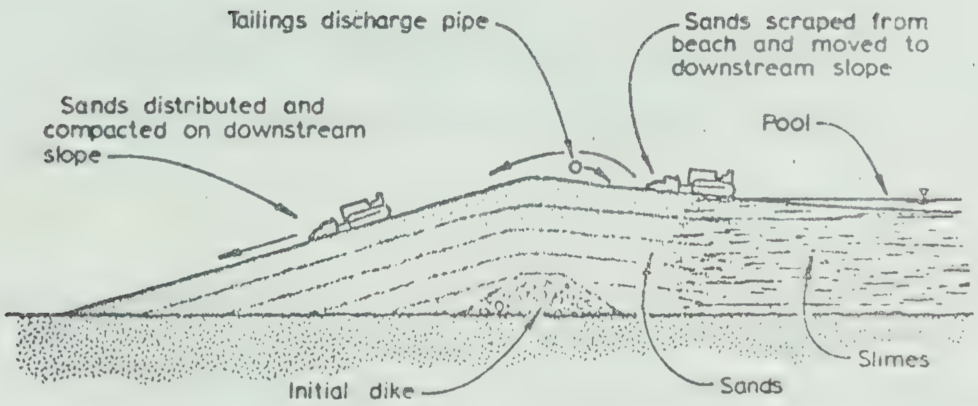
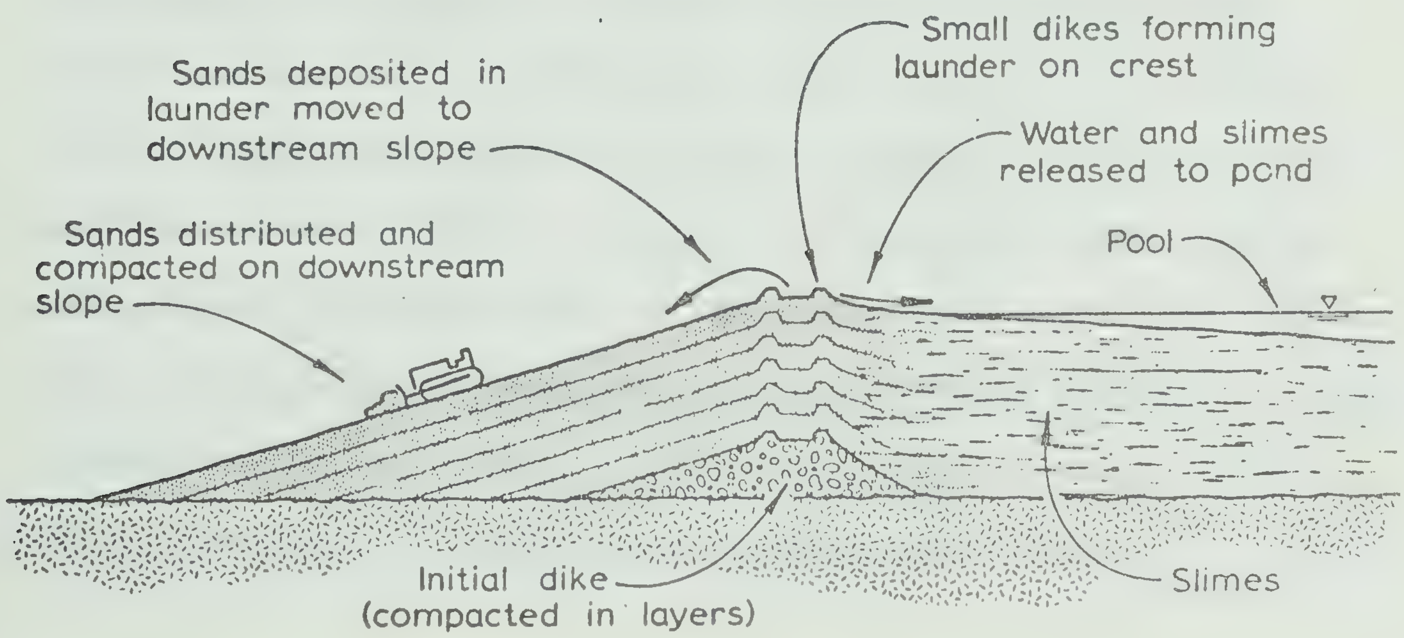


Figure 2.7 Homogeneous Dam With a
Chimney Drain



(After Casagrande and MacIver, 1971)

Figure 2.8 Separation of Sand by Spigotting in Centreline Construction



(After Casagrande and MacIver, 1971)

Figure 2.9 Separation of Sand By Sluicing Technique

CHAPTER III

PHYSICAL PROPERTIES OF TAILINGS AS DETERMINED BY LABORATORY TESTS

3.1 Introduction

It is emphasized in the previous chapter that there is a serious need for a detailed and systematic study of the physical properties of tailings. A relatively extensive program of laboratory testing has been undertaken to rectify this situation; the pertinent details are presented in this chapter. A limited amount of work on the subject at hand has already been reported by other investigators such as Pettibone and Kealy (1971), Guerra (1973), Klohn and Maartman (1973), Hoare (1972) and Hamel and Gunderson (1973). Results of these previous investigations are incorporated in this chapter where applicable. Since some of the materials in this testing program had been classified by cyclones, a brief description of a cyclone classifier and its operation is also included.

3.2 Materials Tested

In November, 1971, five, typically large, mining companies in Canada were requested to participate in the proposed testing program by supplying samples of their tailings materials. Table 3.1 includes a brief description

of location, ore processed and mining operation, and method of separating sand from tailings for all mines that supplied materials in response to this request. Mineralogical composition of ores at the various mines is presented in Table 3.2.

In addition to the materials described above, tests are also performed on additional materials obtained at Craigmont, Bethlehem and Brenda mines during subsequent field investigations. Tailings materials studied in this investigation largely fall into three categories:

i) tailings - total waste material from a concentration plant,

ii) sands - coarse fraction separated from tailings by cyclones, and

iii) slimes - fine materials left behind after removal of sands.

Where necessary for comparison purposes, standard Ottawa sand and sand from the Fraser River in British Columbia are also included in the testing program.

3.3 Brief Description of the Cyclone

Various methods of separating sand from tailings are described in Chapter II. Of all the methods discussed, it is indicated that the cyclone (or more precisely the hydrocyclone) shows the most promise. Therefore, sands

separated by cyclone classifiers are studied in this investigation. Design and operation of the cyclone are covered in great detail by Bradley (1965). In order to fully understand the implications of various stages of cycloning used in classifying sands, however, a brief description of the cyclone and its operation is included here.

Cyclones are frequently used in mining operations as thickeners or classifiers. In separation of sand from tailings, the cyclone works as a combination thickener-classifier for simultaneous dewatering and desliming to obtain free draining sands without excess water.

Figure 3.1 illustrates principal features of the cyclone. Tailings slurry is injected into the cyclone chamber under pressure and in a tangential direction. The tangential injection results in utilizing the fluid pressure to create rotational motion of the injected fluid. This rotational motion causes relative movement of particles suspended in the fluid, thus permitting separation of these particles either from one another or from the fluid.

The cyclone chamber at the point of entry is usually cylindrical. It can remain cylindrical over its entire length though it is more usual for it to become conical. The outlet for the bulk of the fluid is usually

located near or on the axis of the chamber such that the rotating fluid is forced to spiral towards the center to escape. The rotational motion of the fluid has, therefore, an inward radial component. Particles of a suspended material consequently have two opposing forces acting on them - one in an outward radial direction due to the centrifugal acceleration, and the other in an inward radial direction due to the drag force of the inward moving fluid.

The magnitude of these forces is dependent on the physical properties of both the fluid and the suspended material. Through proper consideration of these properties, separation of one material from another or of a single material from the fluid can be achieved.

In either of the above cases, one product moves radially outwards while the other moves radially inwards. The cyclone is therefore equipped with two outlets, one on each end. Both outlets are usually axial although one can be peripheral. The principal features of the cyclone shown in Figure 3.1 are the tangential feed inlet, the main fluid outlet (or overflow), and the peripheral fluid outlet (or underflow). The overflow is taken out through a pipe (known as the vortex finder) which protrudes from the roof of the cylindrical section. The underflow is taken out through an opening in the apex of the conical section. The cyclone is normally sized according to the

diameter of the cylindrical section. For example, a 6-inch cyclone is one with the internal diameter of the cylindrical section equal to 6 inches.

The cyclone is illustrated in Figure 3.1 with its axis vertical. Except for large cyclones operating at low pressures, the position of the axis is immaterial. Gravity plays no part in the separational forces except in these extreme cases.

In the case of tailings, the cyclone forces sand particles to the outside and they are removed through the underflow. Slimes along with most of the water are removed through the overflow. Some of the slimes along with the water in the sands, however, also escape in the underflow. A second stage cycloning is sometimes employed. This may involve further desliming of the sand from the underflow of the first cyclone or recovery of additional sand from the overflow of the first cyclone. In this thesis, the sand when it is refined by two successive cycles of cycloning is termed twice cycloned. The additional sand recovered in the second stage of cycloning is described as secondary, and the sand from the underflow of the first cyclone or when it is subjected to a single cycle of cycloning only is described as primary.

3.4 Tests on Sands

3.4.1 Description of Materials

Figure 3.2 presents grain size distribution curves for all sands included in this study. Tailings sands are angular to very angular in particle shape except for the material from Great Canadian Oil Sands, which is sub-angular to sub-rounded. Figures 3.3 to 3.8 are micro-photographs of the various sands.

For identification purposes, the tailings sands included in this testing program are named after the mines of their origin, such as Brenda sand or Bethleham sand. It should be pointed out, however, that the grain size distribution of a tailings sand at a typical mine can vary, sometimes due to changes in the milling process but more frequently due to the ever changing operating conditions such as rate of feed to the mill, grade of ore being mined, etc. Therefore, the sands included herein are at best nominally representative of the materials produced at these mines. These comments also apply to the tailings and slimes materials described in a later section of this chapter.

3.4.2 Relative Density Determinations

To minimize danger of failure by liquefaction, a minimum relative density is usually specified for compaction control in the construction of tailings dams as discussed in the previous chapter. According to ASTM Definitions of

Terms and Symbols Relating to Soil and Rock Mechanics (D653-67), relative density is defined as

$$D_d(\%) = \frac{e_{max} - e_o}{e_{max} - e_{min}} \times 100 \qquad \text{....(3.1)}$$

where e_{max} is void ratio at minimum density,
e_{min} is void ratio at maximum density, and
e_o is a given void ratio (in situ or of a laboratory sample).

The determination of the relative density of any soil mass or soil specimen, therefore, requires three density determinations. Tests have been performed, as part of this study, to determine maximum and minimum densities for sands included in this program; the results are presented in Table 3.3. In situ density determinations carried out in this study are discussed in the next chapter. All maximum and minimum density tests have been performed according to the procedures described in ASTM Test for Relative Density of Cohesionless Soils (D2049-69). Standard Proctor Compaction Tests (ASTM D698-70) are also performed on some of the sands and the results are included in Table 3.3. Pertinent results reported by other investigators, where ASTM Standard Procedures had been used to carry out the tests, are also presented. A review of the results indicates that there is a much larger range in void ratios

at maximum and minimum densities in the case of the highly angular tailings sands (.39 to .56) as compared to that for the rounded Ottawa Sand (.21). This is in general agreement with the recently reported work of Holubec and D'Appolonia (1973) entitled, "Effect of Particle Shape On the Engineering Properties of Granular Soils."

3.4.3. Permeability Measurements

3.4.3.1 General

The subject of permeability of tailings sands is a very basic one to the safe and economical design of tailings dams. There are two aspects of this subject that require consideration:

i) Minimum coefficient of permeability acceptable for a sand to be used in the construction of a tailings dam, and

ii) Influence of void ratio, grain size distribution and amount of fines (-#200 material) on the coefficient of permeability of a tailings sand.

We will concern ourselves herein with the second item only; the first item will be dealt with in a later chapter of this thesis.

3.4.3.2 Previous Studies

Bates and Wayment (1967) report work carried out

at the Spokane Mining Research Centre, U.S. Bureau of Mines. The work is based on a multivariable least-squares regression analysis of 135 separate permeability tests on mill tailings. It is indicated that coefficient of permeability of a tailings material can be estimated if density, specific gravity, and grain size distribution are known. The regression equation is as follows:

$$L_n K_{20} = 11.02 + 2.912\, L_n \{(VR)\ (D_{10})\} - .085\, L_n(VR) L_n(C_u) + .194(VR)\ (C_u) - 56.5\ (D_{10})(D_{50}) \qquad \ldots\ldots(3.2)$$

where K_{20} = coefficient of permeability, in inches per hour at $20^{o}C$,

VR = void ratio, dimensionless,

D_{10} = effective size (diameter in millimeters which corresponds to the 10 percent-finer-than-size on the grain size distribution curve),

C_u = coefficient of uniformity (D_{60}/D_{10}),

D_{50} = average size (diameter in millimeters of the 50 percent-finer-than grain size),

D_{60} = diameter in millimeters of the 60 percent-finer-than grain size, and

L_n = natural logarithm (log to the base e).

The authors have presented Figure 3.9 in support of the accuracy of their above regression equation. It is noteworthy, at this time, that the calculated values

are consistently higher than the measured values particularly in the range where permeability varies between 1 to 10 inches per hour. Typically, this is the range of values for tailings sands. We will discuss this issue further in a later section of this chapter.

The test procedure followed in performing permeability tests included in the above regression analysis has been described by Wayment and Nicholson (1964) as "A Proposed Modified Percolation-Rate Test for Use in Physical Property Testing of Mine Backfill". A brief description of sample preparation and the procedure follows.

A known amount of ovendried material is mixed with a given amount of water to form slurry which is heated to a light boil for deairing purposes. After allowing the slurry to cool, it is remixed and poured into a permeameter using wash bottle if necessary to complete the material transfer. The permeameter is then filled with water and attached to a constant head reservoir. Water is allowed to percolate through the specimen under a constant head condition. After a steady state is reached, the rate of flow of water is measured and the coefficient of permeability computed. Tests are performed at various void ratios so that permeability-void ratio relationship can be established for a given material.

Initially, apparatus and test procedure described above were duplicated in the soil mechanics laboratories at the University of Alberta. The test results were, however, anything but encouraging. Placing of sand slurry into the permeameter resulted in segregation of materials in that a thin layer of fines formed at the top of the test specimen. A test performed on twice cycloned Brenda Sand gave a permeability value of 6.5×10^{-4} centimeters per second. However, after the top thin layer of fines (approximately 2 millimeters thick) was removed, the test gave a permeability value of 1.1×10^{-2} centimeters per second. Neither of these values can be considered to represent the permeability of the sand in question.

In view of the above difficulties, the test procedure was abandoned. Development of a new technique in which segregation of material did not occur was, therefore, necessary.

3.4.3.3 <u>Test Procedure Adopted in this Study</u>

In the test procedure adopted in this study, the ovendried material is placed directly in the permeameter. Porous discs are provided at the bottom as well as the top of the specimen. Once the porous discs are in place, the test specimen is allowed to take in water from the bottom under a relatively small head until free water is noticed above the top porous disc. At this stage, the permeameter

is filled with water and attached to a constant head reservoir. From then on, the test procedure is essentially the same as followed in the previous method. At the same void ratio, the permeability value measured for the twice cycloned sand cited in the previous section is 6.3×10^{-3} centimeters per second.

3.4.3.4 Presentation of Results

Results of constant head permeability tests performed on all tailings sands included in this program are presented in Figure 3.10. The permeability values are plotted as a function of void ratio. It should be noted that by drawing best fitting straight lines through the results presented, permeability values can be estimated at void ratios other than those used in the tests.

Permeability values are also estimated using the regression equation presented in Section 3.4.3.2. Figure 3.11 is a plot of measured versus calculated permeability values for the sands included in the program. Here again, it should be noted that the calculated values are consistently higher than those measured.

In order to examine if permeability of a material can be correlated with the amount of fines (-#200 fraction) present, the test results are presented in an appropriate form in Figure 3.12. The permeability values plotted are

those at a void ratio of 0.9 for materials tested in this study and the test conditions for results of the previous investigations are as described in Figure 3.12. A value of void ratio of 0.9 is selected because it is representative of the initial densities usually obtained in hydraulic placement of tailings sands. A review of the results in Figure 3.12 appears to indicate no definite trend of any correlation between permeability and percent of fines present in the material.

The above permeability values are again plotted in Figure 3.13 but, this time, as a function of D_{10}, (where D_{10} has the usual meaning). The straight line representing the empirical correlation of $K = D_{10}^2$* based on the work of Hazen (1892) is also plotted in this figure.

3.4.3.5 Discussion and Conclusions

The calculated values from the regression equation suggested by Bates and Wayment are generally higher than those measured in this testing program. It is thought that

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* Hazen's formula is usually written as $K = 100D_{10}^2$ when D_{10} is the effective size in centimeters or as $K = D_{10}^2$ when D_{10} is in millimeters.

this discrepancy is the result of either an inherent weakness in the regression equation as discussed in Section 3.4.3.2 or the differences in degrees of saturation of the test specimens due to different testing procedures employed in the two testing programs or both. It is recognized that the new test procedure adopted in this study perhaps results in degree of saturation somewhat lower than 100% in the test specimen. It can be argued, however, that the assessment of permeability of a tailings sand is required not only in a fully saturated condition but also under other conditions when the material may not be fully saturated such as on rewetting. Furthermore, the values obtained from the tests are on the conservative side.

A review of the results presented in Figure 3.13 indicates that the permeability of a tailings sand can be estimated with reasonable accuracy from Hazen's empirical formula $K = D_{10}^2$ at a void ratio of 0.9 which is approximately equivalent to a relative density in the range of 40% to 50% for deslimed tailings sands.

3.4.4 Shear Strength Measurements

3.4.4.1 General

Shear strength of tailings sands can be expressed by Coulomb's well known equation

$$S = C' + \sigma' \tan \phi' \qquad .....(3.3)$$

where S = shear strength,

σ' = normal effective stress,

ϕ' = angle of shearing resistance in terms of effective stress, and

$c' = 0$.

Or, the above equation may be rewritten as

$$S = \sigma' \tan \phi' \qquad \text{..... (3.3a)}$$

Generally speaking, ϕ' is a function of the grain size distribution, angularity and mineralogy of sand particles, density of the sand and the pressure range applicable to the situation at hand. For a given sand, however, the number of variables is reduced to two only - the density and the pressure range. This is well illustrated in Figure 3.14 which gives typical Mohr failure envelopes for dense and loose sands (Bishop, 1971). Similar results are illustrated in Figure 3.15 for a tailings sand (Klohn and Maartman, 1973).

3.4.4.2 Results of Triaxial Tests

Early in the program, a preliminary set of drained triaxial tests was performed on various tailings sands. The results are presented in Table 3.4. The tests were carried out on small specimens (1.5 inches in diameter and 3.0 inches long) in a conventional triaxial equipment. The tests were performed in a strain controlled manner at a strain rate of .005 inches per minute.

The purpose of the tests was to establish a correlation between ϕ' and density at various confining pressures. However, density measurements of test specimens in the above testing technique were suspect because even small errors in measuring the diameter and length of a test specimen appeared to result in a significant error in the calculated volume and hence, the density. The above errors are considered to be inherent in the technique commonly used in forming triaxial specimens of sand because of resulting membrane penetration. In the above testing, the problem was further aggravated due to the small size of the specimens. Any further triaxial testing was, therefore, abandoned.

3.4.4.3 Results of Direct Shear Tests

All further testing has been carried out in a direct shear box in which more accurate density measurements are possible. Tests are performed on two typical tailings sands - one (Brenda Sand) typical of the highly angular materials and the other Great Canadian Oil Sands (GCOS) material which is sub-angular to sub-rounded. For comparison purposes, standard Ottawa sand is also included in this testing program.

Except for GCOS sand which is discussed later, ovendried material is placed in a shear box of known volume and then allowed to saturate prior to application of normal load. Tests are performed at various normal pressures

ranging from 6.7 psi to 120 psi. At each normal pressure, tests are performed at 2 to 3 different densities at a rate of strain of .007 inches per minute.

In the case of GCOS sand, initially ovendried material was placed in the shear box and allowed to saturate by flooding the reservoir around the shear box in a manner similar to that employed for other sands. Due to the oil coating on the particles, however, the sand would not saturate. In fact, except for the thin peripheral zone, most of the material in the specimen remained dry. Thus, in all subsequent testing the sand has been premixed with water and placed in a wet condition. In this procedure, it is not feasible to place sand in loose condition and therefore tests are performed at relatively high densities only.

The direct shear data for the Brenda sand are presented in Figure 3.16 in the form of plots of peak ϕ' versus dry density (after application of normal load) for each normal stress selected for testing. Figure 3.17 illustrates graphs of peak ϕ' versus normal stress at selected densities. These plots are generated on the basis of the data presented in Figure 3.16. Data for the other two sands are presented in Figures 3.18 to 3.21.

In the case of Brenda and Ottawa sands, direct

shear data at loose densities (approximately at zero relative density), are plotted in the more conventional manner in Figure 3.22.

3.4.4.4 Discussion of Results

A review of the direct shear data presented indicates that ø' is a function of the normal stress and the test density (at the start of shearing) up to a value of the normal stress of about 50 or 60 psi. At higher stress levels, ø' appears to be essentially independent of the normal stress. It is interesting to note that in the case of the tests performed at extremely loose densities, Figure 3.22 gives relatively constant values of ø' independent of the stress level. Hoare (1972) reports similar results for a tailings sand (with 17% passing the #200 sieve) when tested in a loose and ovendried condition. The ø' obtained was 33.5 degrees. This is comparable to the ø' of 33.7 degrees obtained from loose Brenda Sand (Figure 3.22). On the basis of these results it can be concluded that the tests at loose densities give lower bound values of ø'.

It should be noted that ø' for the highly angular Brenda tailings sand is generally 5 to 6 degrees higher in value than that for the rounded Ottawa sand at all densities and stress levels.

3.4.5 Consolidation Tests

In order to study compressibility characteristics of tailings sands, consolidation tests have been performed on three typical materials. Sand is placed in the odeometer ring in an ovendried and loose condition (zero relative density). Except for the GCOS sand, the other two sands are then allowed to saturate before starting the test. In the case of GCOS sand, the test is performed on the ovendried material and no attempt is made to saturate the sand for reasons given in Section 3.4.4.3. Graphs of void ratio versus logarithm of consolidation pressure applied during the tests are presented in Figure 3.23.

3.5 Tests on Slimes and Tailings

3.5.1 Description of Materials

As discussed previously in Section 3.2, tailings are the total waste material left behind after the removal of mineral concentrates and are transported as such to the disposal area. Grain size distribution curves for typical tailings from various mining operations are presented in Figure 3.24. Slimes are the finer fraction of tailings left behind after the removal of sands. Grain size distribution curves for tailings and slimes materials included in this study are shown in Figure 3.25.

Tailings and slimes can generally be classified as sandy silts or silts and range from low to non plastic

in nature. Plasticity results for typical tailings and slimes tested in this program are presented in Table 3.5.

3.5.2 Direct Shear Tests

Direct shear tests have been performed on two typical slimes samples recovered from about the middle of the Brenda and Bethlehem tailings ponds. Material is placed in the shear box in the form of a slurry and allowed to consolidate under the desired normal stress for the test. The specimen is then sheared in a normally consolidated condition at a constant rate of strain of .0045 inches per minute. This rate of strain is slow enough to give pore pressure dissipation of over 95% according to Bishop and Henkel (1962) which is common practice for direct shear tests. The results are shown plotted in Figures 3.26 and 3.27. For all practical purposes, the values of 30.5 degrees and 32.0 degrees for ϕ' obtained are comparable to those of loose tailings sand as discussed previously in Section 3.4.4.4.

3.5.3 Oedometer Tests

3.5.3.1 General

To investigate the compressibility characteristics, the rate of drainage and the rate of progress of consolidation of slimes and tailings materials disposed of in the storage ponds, a series of oedometer tests have been performed. In the oedometer, in addition to the conventional

consolidation tests, constant head permeability tests are performed on the specimens at selected intervals in the loading sequence. Seven different materials are tested from four different mining properties. Duplicate tests are performed on all samples.

3.5.3.2 Equipment and Test Procedure

Testing has been carried out in an Anteus consolidometer. This equipment has been described in detail by Lowe et al (1964). Briefly, this equipment differs from the more conventional consolidation apparatus in that it has a pneumatic loading mechanism and the specimen is enclosed in a chamber so that back pressure can be applied. With the back pressure arrangement, a permeability test can be performed on the specimen at any stage during the consolidation test.

Material is thoroughly mixed with water to form a thin slurry and poured into the oedometer ring. It is then allowed to consolidate under its own weight before placing the top porous stone. Initially, a very small load is applied to the specimen and the test is continued from then on in a conventional manner except that constant head permeability tests are performed at selected intervals.

3.5.3.3 Presentation of Results

Results from the odeometer tests are presented in the following four forms for each material tested:

i) void ratio versus log K,

ii) void ratio versus log P,

iii) void ratio versus log m_v, and

iv) void ratio versus log C_v - generated on the basis of (ii) and (iii)

where P = consolidation pressure,

K = coefficient of permeability,

m_v = coefficient of volume compressibility, and

C_v = coefficient of consolidation.

Values of m_v and C_v are computed from the following commonly known equations in soil mechanics,

$$m_v = \frac{\Delta e}{(1 + e_o) \quad \Delta P}, \text{ and} \qquad .....(3.4)$$

$$C_v = \frac{K}{m_v \quad \gamma_w} \qquad .....(3.5)$$

Figure 3.28 presents void ratio versus log K for all materials included in this program. The other results are presented in Figures 3.29 to 3.31.

3.5.3.4 Discussion of Results

An examination of Figure 3.28 indicates that coefficient of permeability (K) of slimes included in this program varies over a fairly large range. For example, at a void ratio of 1.0, K ranges from 5.0×10^{-5} centimeters per second (cm/sec) for the non plastic Copper Cliff material to 1.2×10^{-6} cm/sec for the low plastic material recovered from the Bethlehem tailings pond. At the same void ratio, for the two tailings materials tested, permeability values range from 3.0×10^{-4} to 3.0×10^{-5} cm/sec. These values are within the range of 10^{-3} to 10^{-8} cm/sec reported for fly ash and chemical waste by Casagrande and MacIver (1971). Blight (1969) reports an average value of 1.0×10^{-5} cm/sec for slimes and tailings from the gold mines in South Africa.

A review of the e- log p curves in Figure 3.29 indicates that typically they are non-linear at very low stresses. At stresses greater than approximately 0.1 tsf (tons per square foot), the curves are essentially linear giving a C_c (Compression Index) ranging from .13 to .25. These values are quite typical of those usually obtained for natural silts.

It is interesting to note that m_v varies over a range of 2 to 3 orders of magnitude for the range of void ratios encountered in the odeometer tests (Figure 3.30).

For the same range of void ratios, K, generally, appears to vary over a range of only one order of magnitude (Figure 3.28). Since C_v values are computed from Equation (3.5) this results in C_v varying over a range of 1 to 2 orders of magnitude (Figure 3.31) with the lowest values being at the initial high void ratios. At these high void ratios, the typical values of C_v for the slimes from copper mines in British Columbia included in this program appear to be in the range of 10^{-2} to 10^{-3} cm^2/sec (centimeters squared per second). Blight (1969) reports a value of C_v for slimes from gold mines in South Africa of the order of 1.0×10^{-1} cm^2/sec. At high void ratios which are likely to exist in the slimes deposits, at least initially, Blight's figures are comparable to those obtained for the relatively coarse and non-plastic Copper Cliff material. The values cited previously for the copper slimes from British Columbia, on the other hand, are 1 to 2 orders of magnitude lower than those reported by Blight (1969). The values of C_v for these copper slimes are, however, entirely comparable to those reported by Lambe and Whitman (1969) for soils with liquid limits of about 30 percent.

3.5.4 Sedimentation Tests

3.5.4.1 Purpose of Tests

Slimes are usually discharged into storage ponds in a thin slurry containing approximately one-third to one-quarter solids on the basis of weight. In soil mechanics

terminology, this means that the slurry is at a water content of about 200 to 300 percent. It is recognized that the slimes are subjected to two distinct processes after entering a storage pond (Department of Energy Mines and Resources, 1972). The first process is the initial sedimentation of solids into a very loose soil and the second is consolidation of the resulting soil, under self weight and the weight of the materials sedimenting subsequently in the on-going process of slimes disposal. It is thought that the sedimentation of material, which has entered the pond at a specific time, occurs rather rapidly. Consolidation of the same material, however, is an on-going process for the operating life of the pond and for many years afterwards. Tests are performed on typical slimes at a water content of 200 percent to determine the possible rate of sedimentation and the initial void ratio of the sediment. Both of these factors are important in assessing the rate of progress of consolidation and the amount of water available for re-use in the concentration plant.

3.5.4.2 Test Procedure

The test procedure followed is essentially the same as that discussed by McRoberts (1973) in his detailed treatment of the sedimentation theory. Briefly, the test procedure involves initially preparing a homogeneous slurry of slimes and water at the desired water content. The slurry is then placed in a glass cylinder. The advance

of the slurry-water interface at the top is then recorded with time. When the distance through which the interface moves is plotted against time elapsed from the start of the test, the early portion of the graph gives a linear plot. The slope of the linear portion of the plot is considered to give the rate of sedimentation.

In the above tests, the specimens are then allowed to consolidate under self weight and final water contents determined. These water contents can be assumed to represent roughly the initial void ratios of the slimes sediment.

3.5.4.3 Test Results

Results of the tests performed on Brenda slimes, and on samples recovered from the Bethlehem and Brenda tailings ponds, are shown plotted in Figure 3.32. As shown the sedimentation rates vary from 1.2×10^{-3} to 2.6×10^{-3} cm/sec. These values are comparable to those reported by McRoberts (1973) for Fort Norman and Devon silts at a water content of 200% (1×10^{-3} to 3.3×10^{-3} cm/sec).

Water contents of the pond samples of slimes after consolidation under self weight are approximately of the order of 100 percent. It is interesting to note that the void ratios recorded for these materials in the odeometer tests are of the order of 1.5 under relatively small consolidation pressures of less than 60 psf (Figure 3.29).

A void ratio of 1.5 is equivalent to a water content of approximately 55 percent for these materials. This large change in the water content from 100 percent under self weight to 55 percent under a small consolidation pressure of 60 psf is quite indicative of the high compressibility of these materials at these high water contents.

TABLE 3.1 Participating Mining Companies

Name of Company	Location of Mine	Daily Throughput Capacity Tons per day	Open Pit or Underground	Type of Ore Mined	Method of Separating Sand
Bethlehem Mines	Highland Valley, B.C.	16,000*	open pit	copper	cyclones
Brenda Mines	Peachland, B.C.	24,000*	open pit	copper - molybdenum	cyclones
Craigmont Mines	Highland Valley, B.C.	5,000*	open pit/ underground	copper (also some iron)	gravity separation (upstream construction)
The International Nickel Company of Canada Ltd.	Sudbury area Ontario	various mines		nickel-copper	cyclones
Great Canadian Oil Sands Ltd.	Ft. McMurray, Alberta	100,000 (45,000 bbl/day)	open pit	tar sands	hydraulic sluicing

* Rated capacity based on figures reported in "The Financial Post Survey of Mines, 1973".

TABLE 3.2 Mineralogy of Ores

Name & Location of Mine	Rock Type	Quartz %	Feldspar Plagioclase %	Feldspar Orthoclase %	Misc.	Remarks
Bethlehem Copper Highland Valley B.C.	Quartz Diorite	15-20	60-65	5-10	10-15% Hornblende and Biotite	
Brenda Mines Peachland, B.C.	Speckled Quartz Diorite	20-25	45-55	10-15	10-15% Hornblende Biotite	
Great Canadian Oil Sands, Alberta	Sandstone	100				
Copper Cliff Ontario Nickel-Copper	Norite					According to Boldt (1967) Sudbury Nickel ores are norites (Gabbro) - main constituents are pyroxene and plagioclase feldspar.
Craigmont Mines B.C.	Diorites and	2-10	50-55		30-32% Hornblende	5 - 10% magnetite
	Andesite	0			50%	35% Andesite 12% Iron Oxide 2-3% Apatite

TABLE 3.3 Results of Minimum & Maximum Density Tests

Material	Gs gm/cc	d_{10} mm	C_u	Minimum Density ASTM (D2049-69) Trials γ d-pcf	Minimum Density Average* γd-pcf	Minimum Density Average* e_{max}	Maximum Density ASTM (D2049-69) γd-Trials (pcf) Dry	Maximum Density γd-Trials (pcf) Saturated	Maximum Density Average * γd-pcf	Maximum Density Average * e_{min}	$e_{max} - e_{min}$	Standard Proctor Max. γ d-pcf	Reference
Ottawa Sand ASTM(C-109)	2.65	.22	1.8	96.0 96.8 96.1	96.3	.72	109.7 109.3		109.5	.51	.21		This Study
Fraser River Sand (-#20)	2.71	.17	1.8	86.2 86.0	86.1	.96	104.4 104.5	100.5	104.5	.60	.34		This Study
**Brenda Sand (72) Primary Cyclone	2.72	.045	4.7	85.3 85.5	85.4	.985	108.2 109.0	-	108.6	.565	.42		This Study
Brenda Sand(71) Twice Cycloned	2.72	.08	2.5	80.5 81.0 81.0	80.8	1.10	101.5	101.3	101.4	.67	.43	94.7	This Study
Bethlehem** Sand (71)	2.70	.08	3.1	81.2 81.4	81.3	1.07	103.0 103.6		103.3	.63	.44	102.2	This Study
Bethlehem Sand (72)	2.70	.045	3.5	75.3 75.4 75.8	75.5	1.23	95.2	100.5 101.7	101.1	.67	.56	96.8	This Study
Craigmont Tailings	3.0	.005	30.0									116.7	This Study
Copper Cliff Sand	3.03	.043	4.0	90.8 90.6 91.0	90.8	1.08		110.5 114.5 111.5	112.2	.68	.40	98.6	This Study
GCOS Sand	2.65	.09	2.2	80.1 80.0	80.0	1.07	94.0	99.0 98.0	98.5	.68	.39	98.0	This Study

TABLE 3.3 Results of Minimum & Maximum Density Tests (cont'd)

Gibraltar Sand	2.70	.075	5.0	81.0	1.08	110.0	.54	.54	Klohn and Maartman
Brenda Sand	2.71	.12	2.5	81.0	1.08	105.0	.61	.47	(1973)

Gs - specific gravity (gm/cc); d_{10} - effective diameter (diameter of 10% passing size); $C_u = \frac{D_{60}}{D_{10}}$ (coefficient of uniformity).

* Values used in subsequent work

** Numbers in parenthesis represent year in which sample was obtained

*** Where results of either dry or saturated tests only are presented, this was based on preliminary tests not reported here.

TABLE 3.4 Summary of Results From Triaxial Tests

Sand Material	$\bar{\sigma}_3$ psi	At the Start of Shearing e	Dd - %	Peak ϕ' assume c'=0
Brenda (71)	28.4	.78	74	40°
Bethlehem (71)	28.4	.75	73	40°
	56.8	.70	84	41°
Copper Cliff	28.4	.82	65	38.3°
	56.8	.77	78	39.5°
GCOS	28.4	.70	95	35°

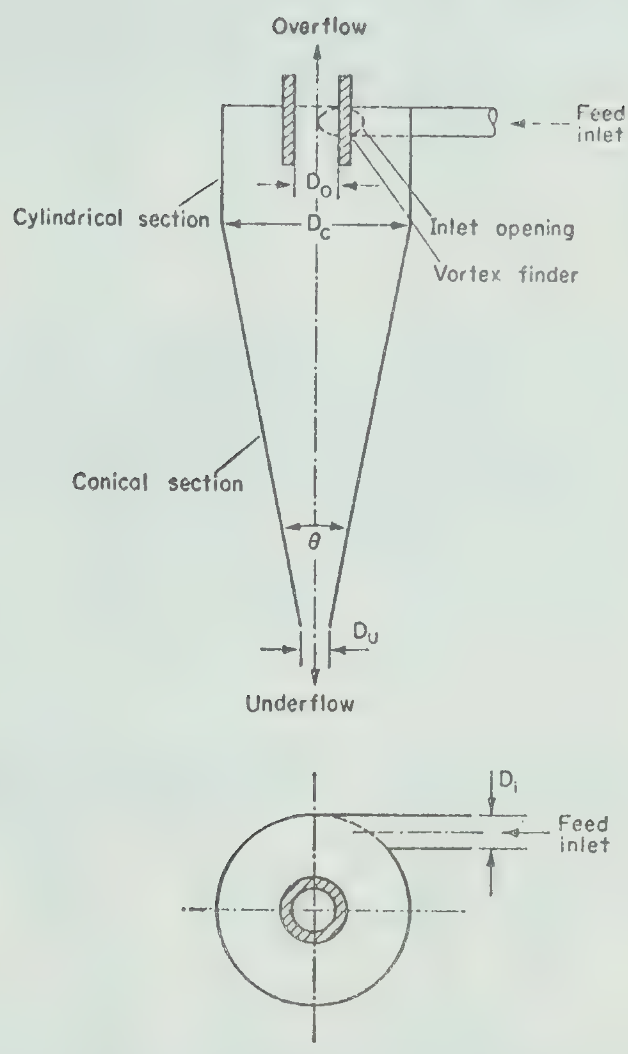
e - void ratio

Dd - Relative Density in %

TABLE 3.5 Plasticity Results for Typical Slimes and Tailings

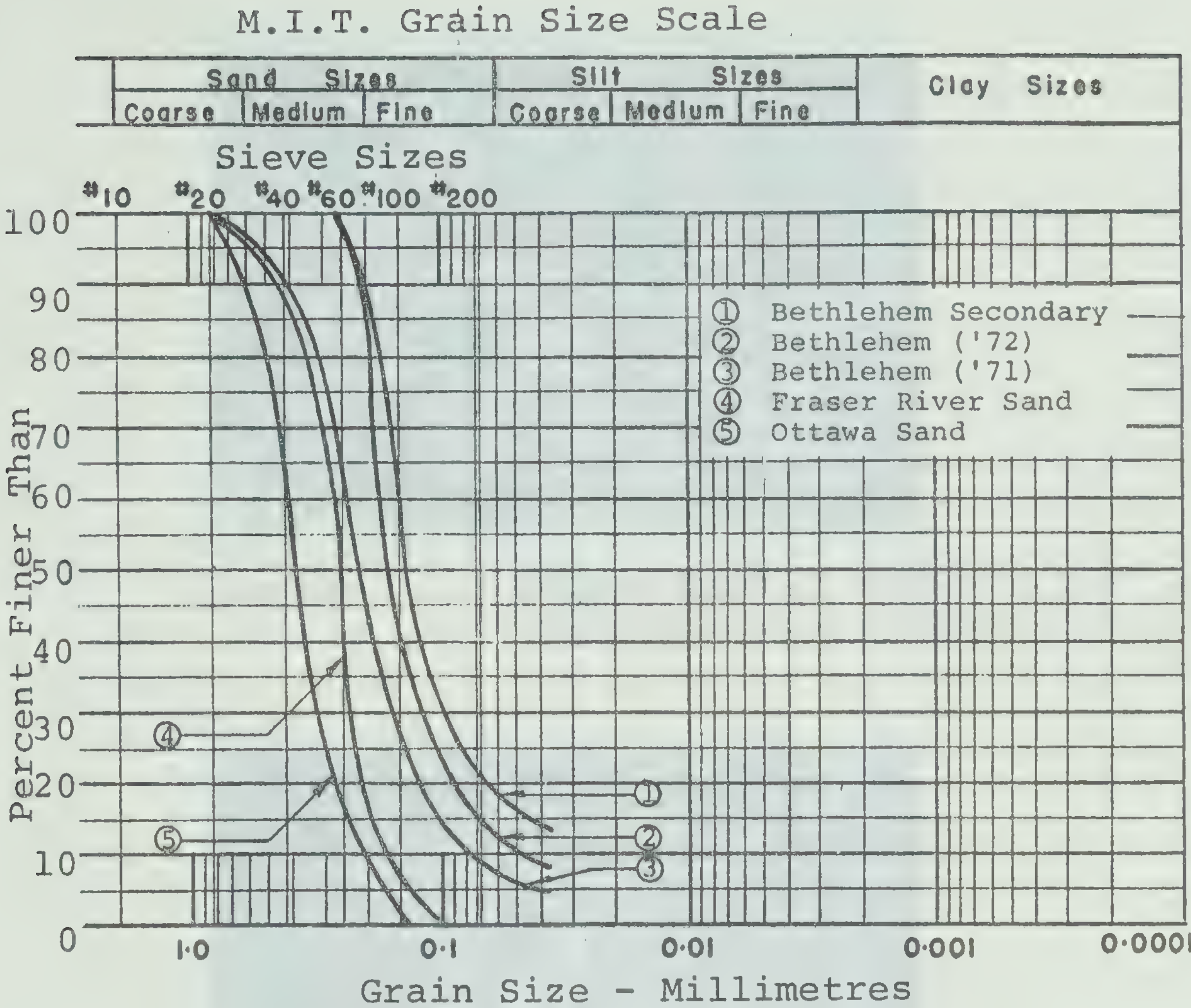
Material	G_s	D_{10} mm	C_u D_{60}/D_{10}	+#200 % Sand	% Clay Sizes M.I.T. Scale	Atterberg Limits Liquid Limit W_L - %	Atterberg Limits Plastic Limit W_p - %
Bethlehem Slimes	2.72	.0007 estimated	25	13	20	30.3	20.0
Brenda Slimes	2.76	.0032	17	30	3	28.2	19.2
Copper Cliff Slimes	2.98	.005	9	27	0	N.A.*	N.A.
Brenda Tailings	2.74	.007	20	56	1	19.8	N.A.
Craigmont Tailings	2.96	.005	28	56	5	23.3	N.A.

* N.A. - Not Applicable (Not able to perform test)



(After Bradley (1965))

Figure 3.1 Principal Features of a Hydrocyclone



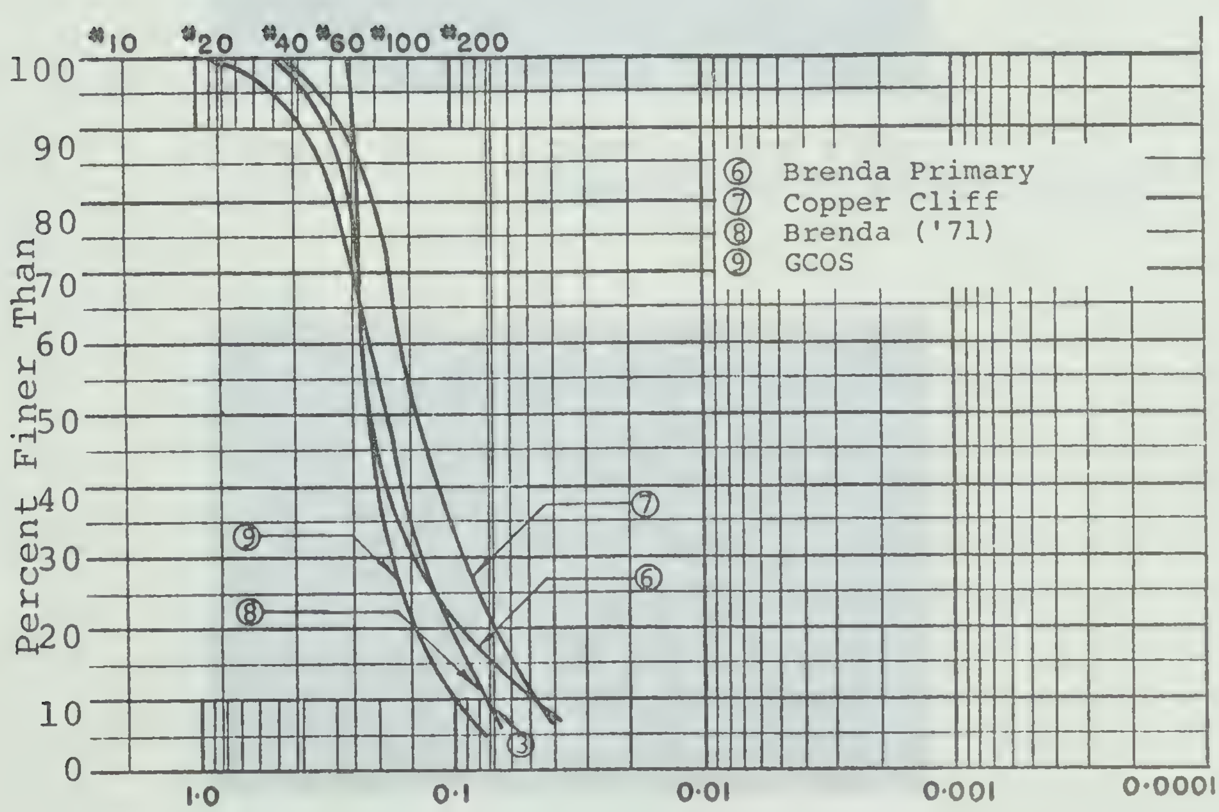


Figure 3.2 Grain Size Curves of Sands Included in This Study

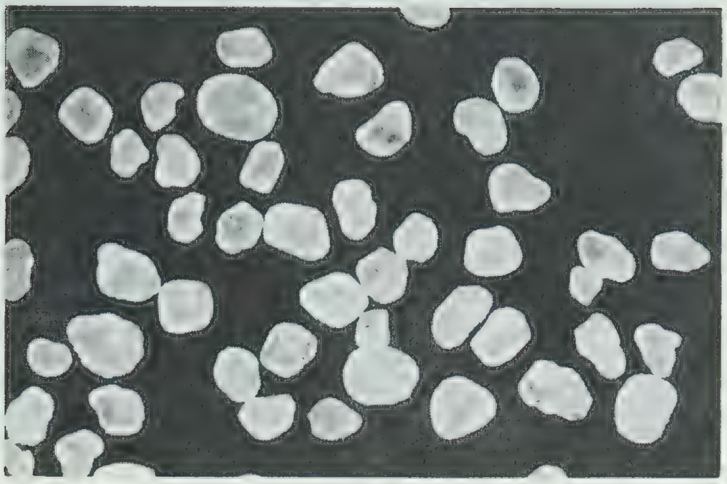


Figure 3.3 Ottawa Sand (c-109)
(Magnification Factor = 10)

Figure 3.4 Fraser River Sand
(Magnification Factor = 10)

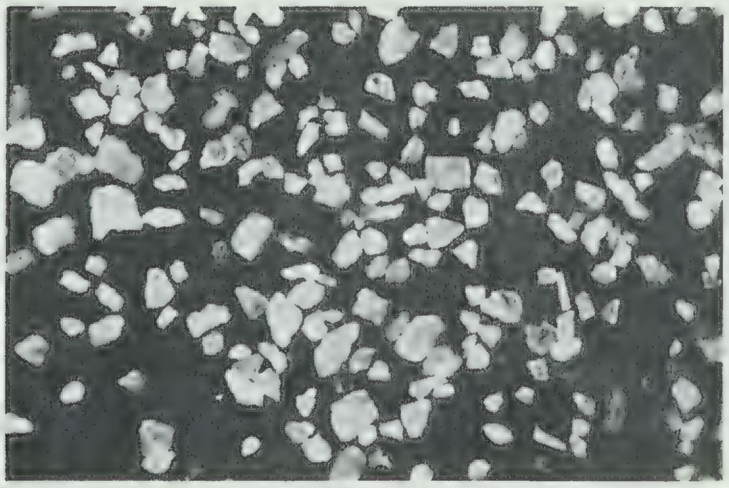


Figure 3.5 Brenda Sand (71)
(Magnification Factor = 10)

Figure 3.6 Bethlehem Sand (71)
(Magnification Factor = 10)

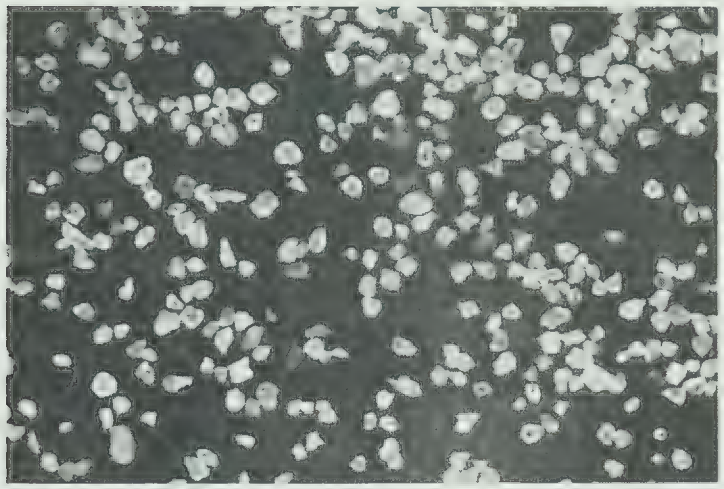


Figure 3.7 G.C.O.S. Sand
(Magnification Factor = 10)

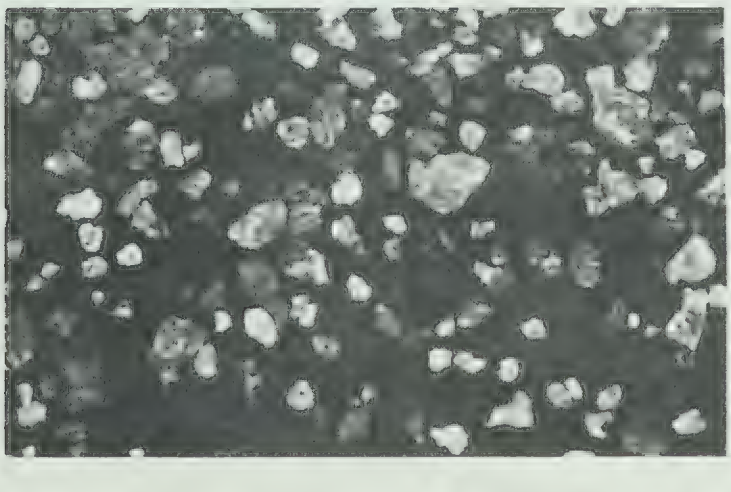
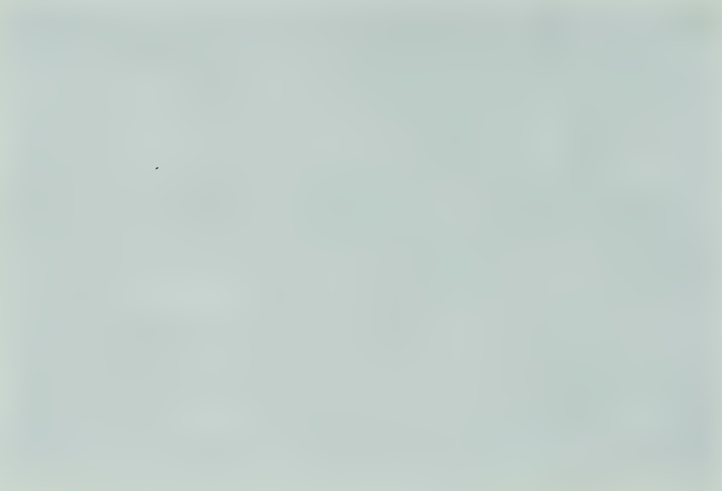


Figure 3.8 CopperCliff Sand
(Magnification Factor = 10)



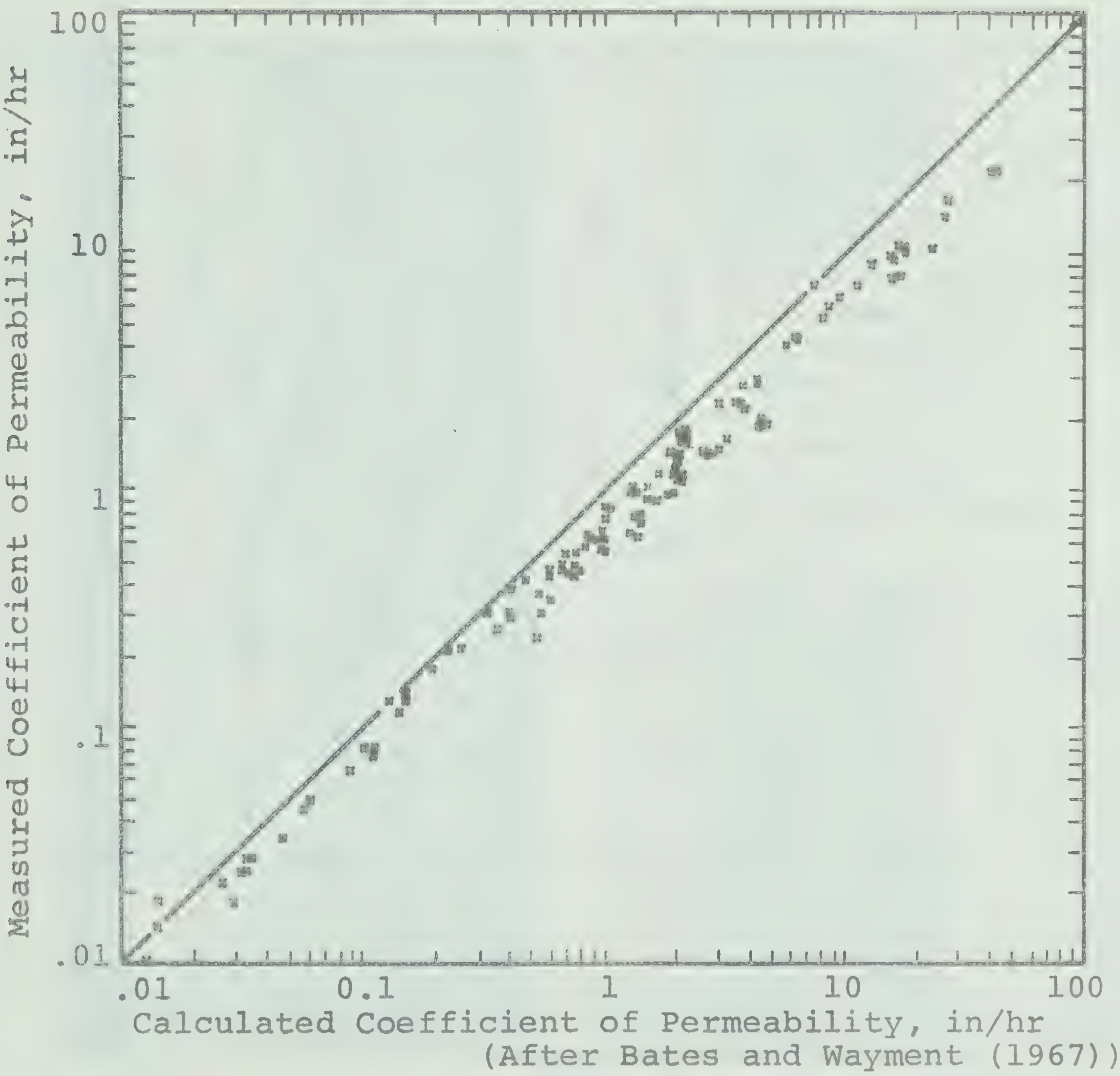


Figure 3.9 Calculated vs Measured Coefficient of Permeability

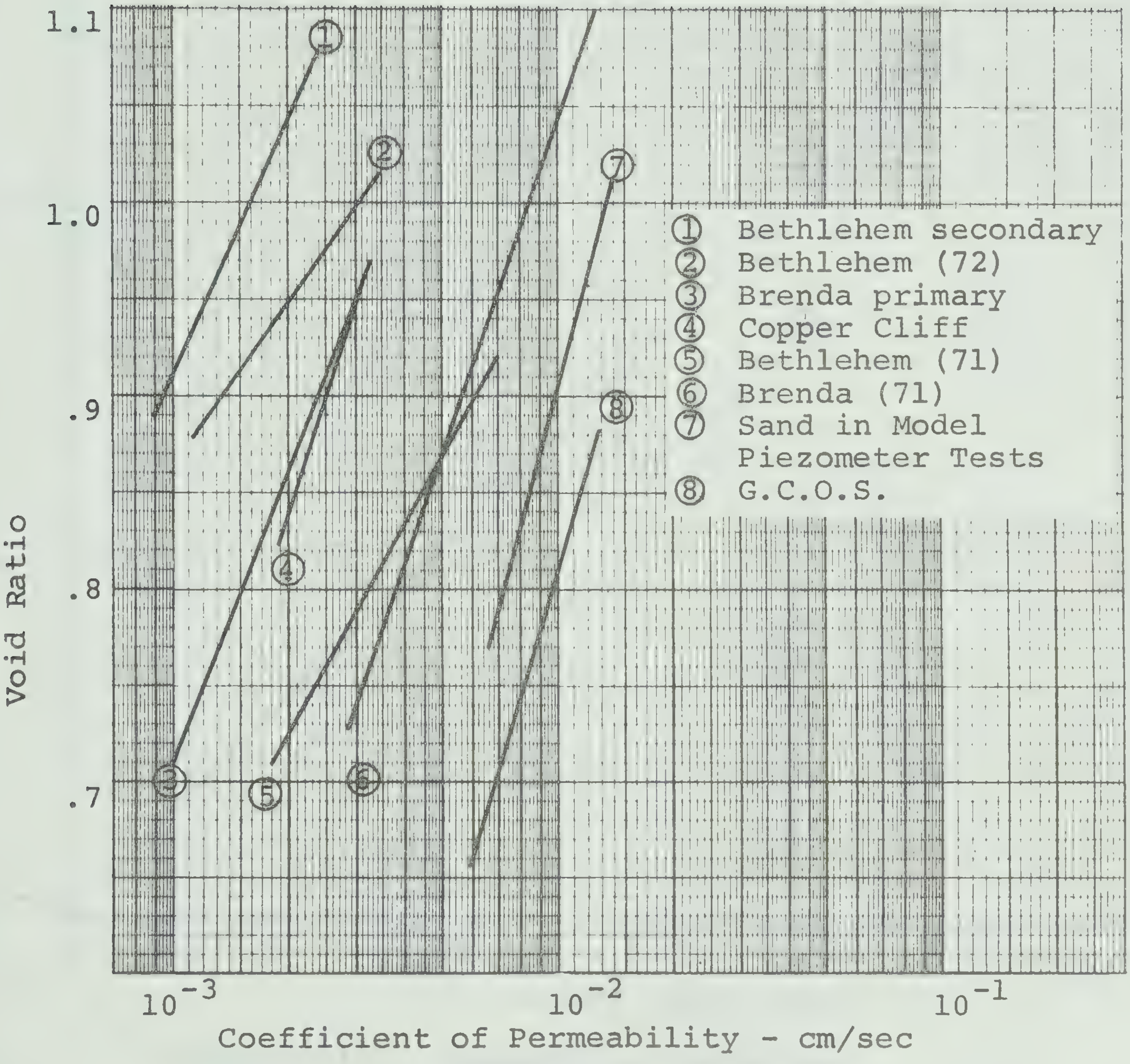


Figure 3.10 Permeability of Tailings Sands

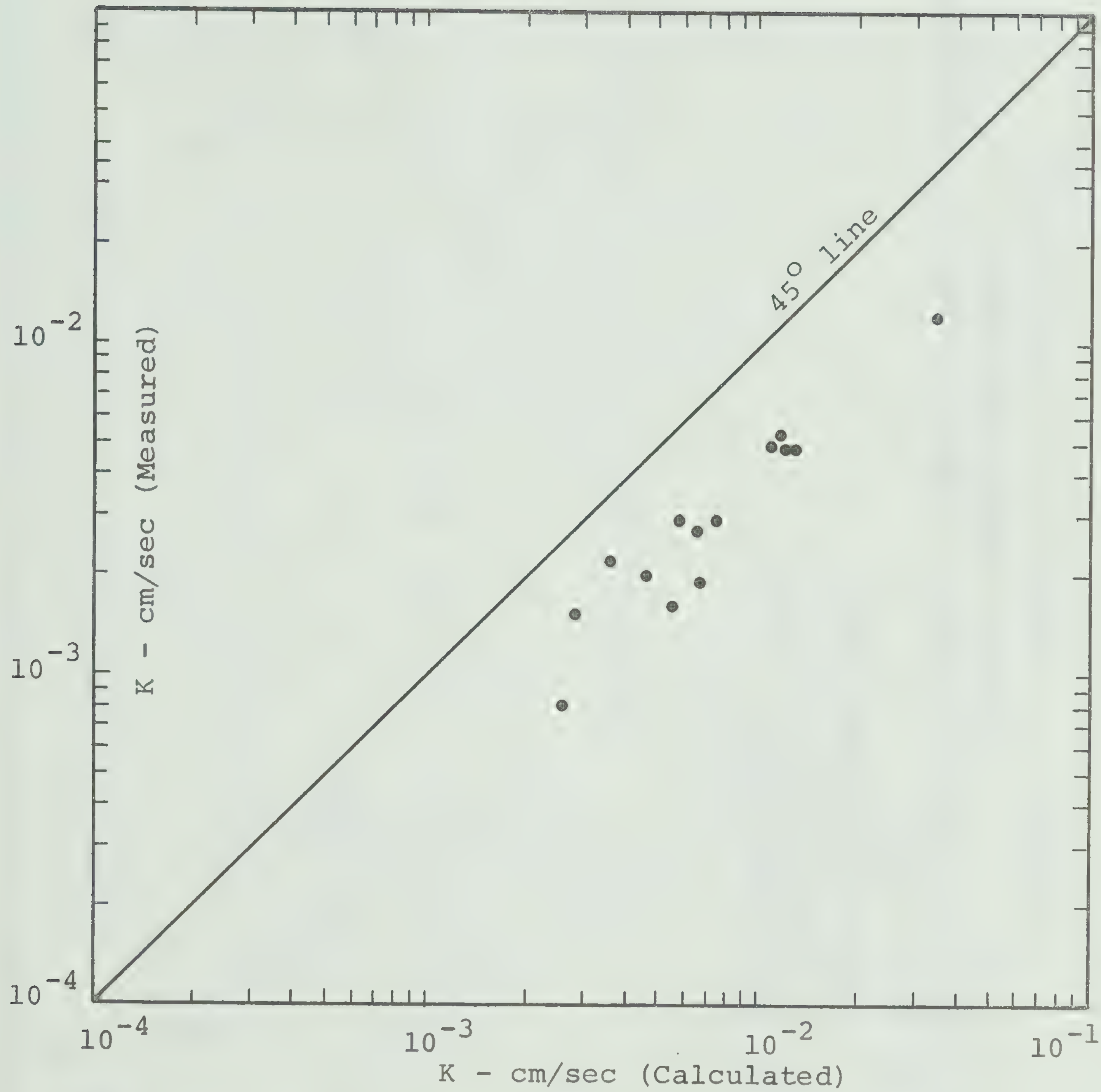


Figure 3.11 Measured vs Calculated Permeabilities

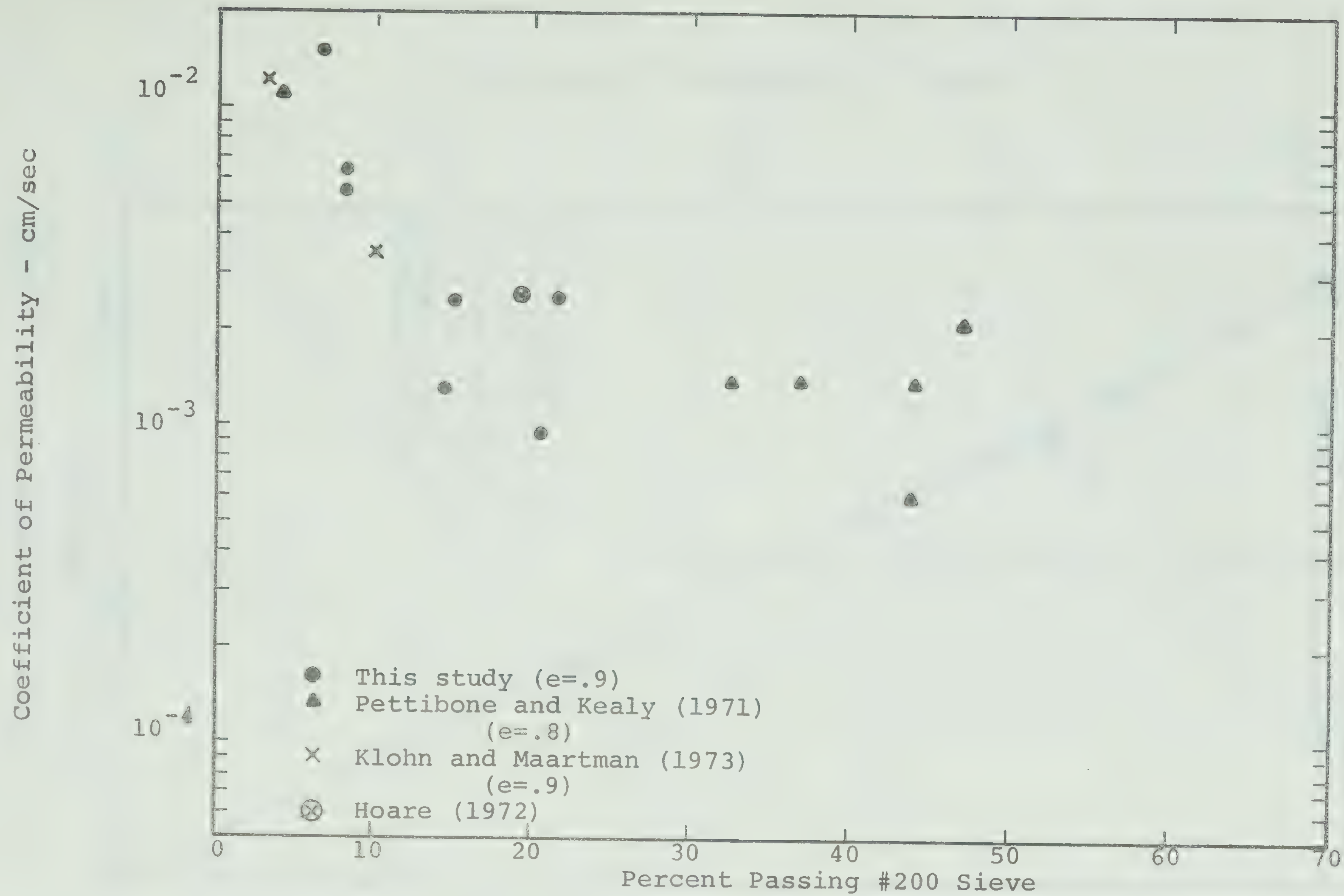


Figure 3.12 Permeability vs Percent Fines

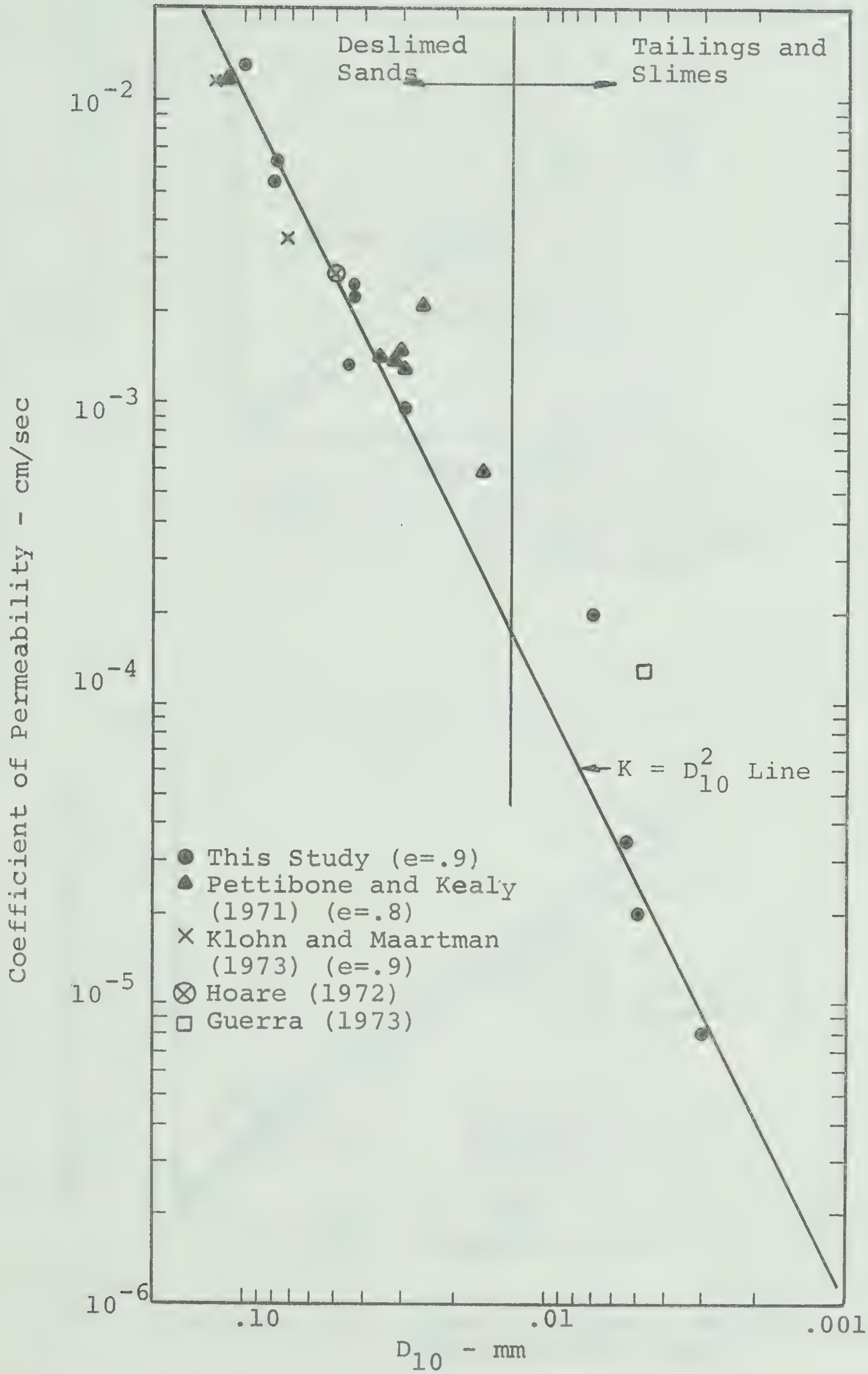
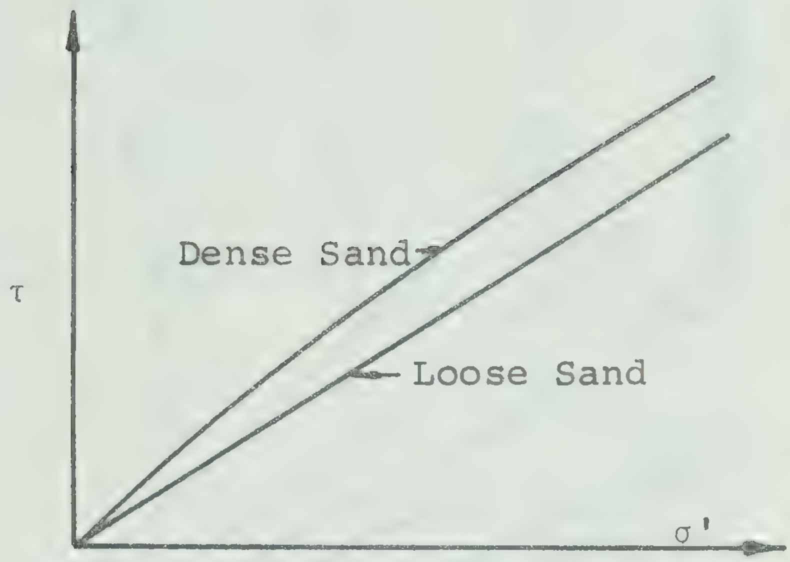


Figure 3.13 Permeability vs D_{10}



(After Bishop, 1971)

Figure 3.14 Typical Mohr Failure Envelopes for Dense and Loose Sand

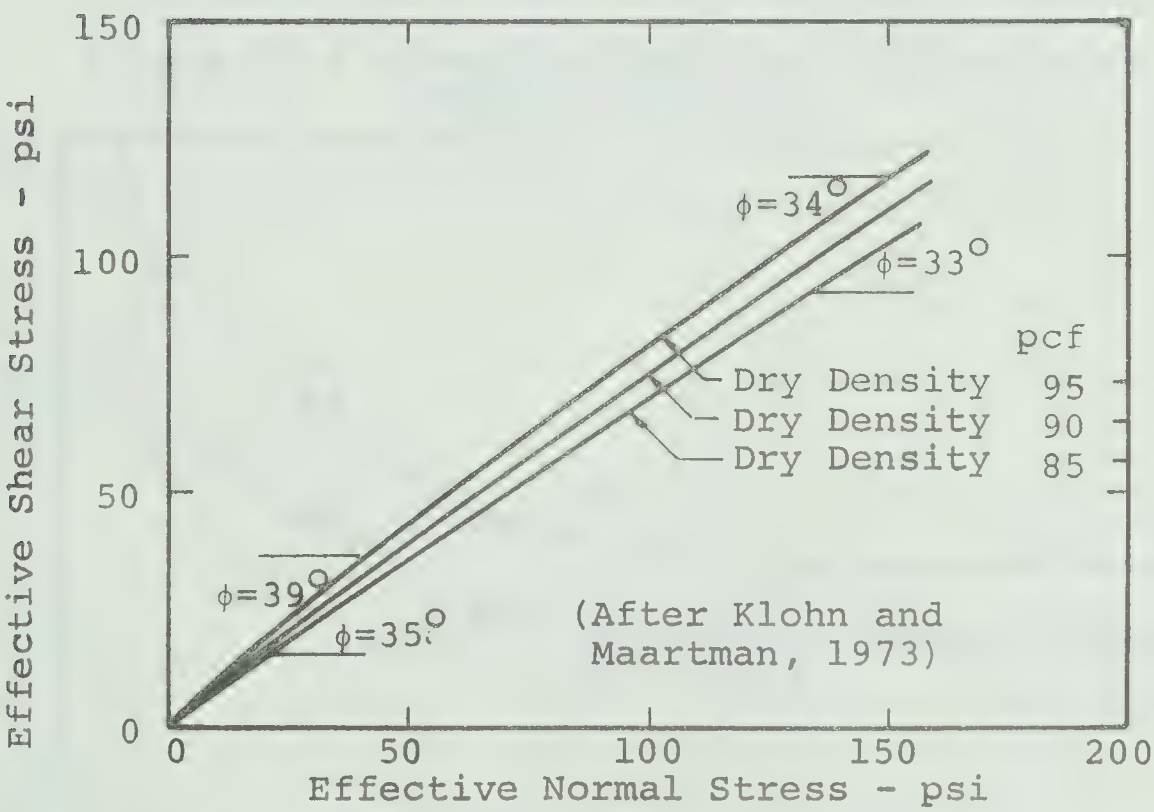


Figure 3.15 Density-Strength Relationships For a Tailings Sand

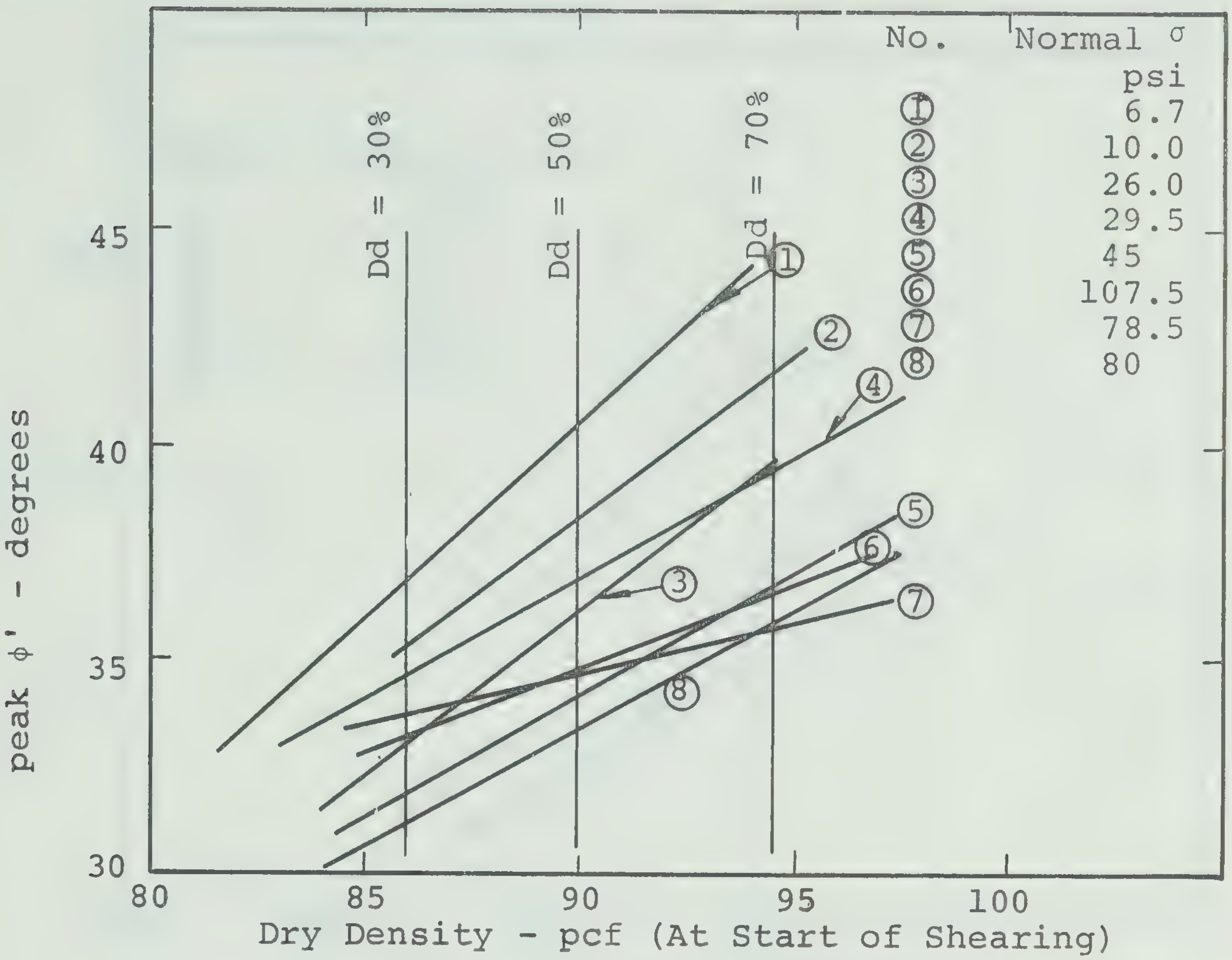


Figure 3.16 Results From Direct Shear Tests Brenda Sand (71)

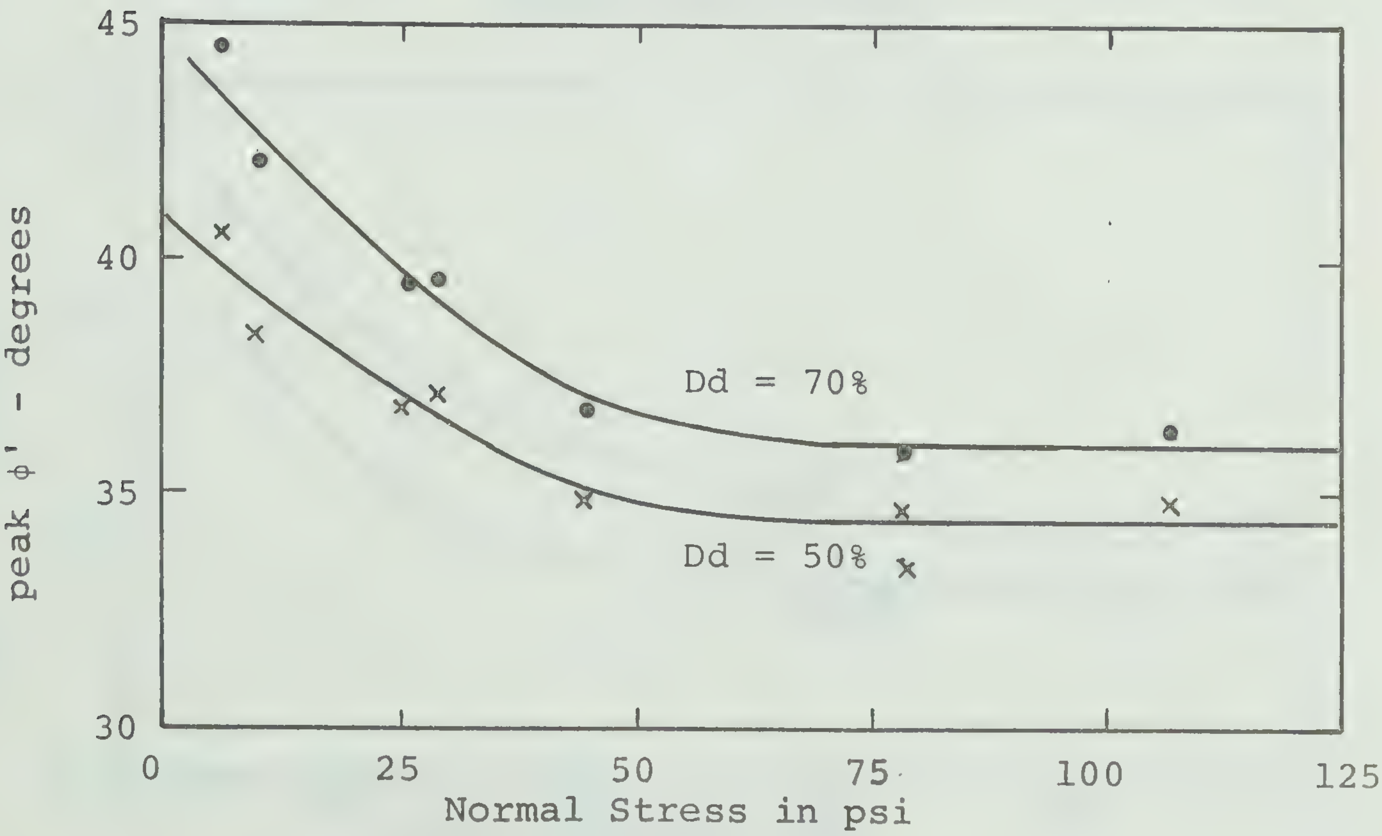


Figure 3.17 Peak φ' vs. Normal Stress (Reference Figure 3.16)

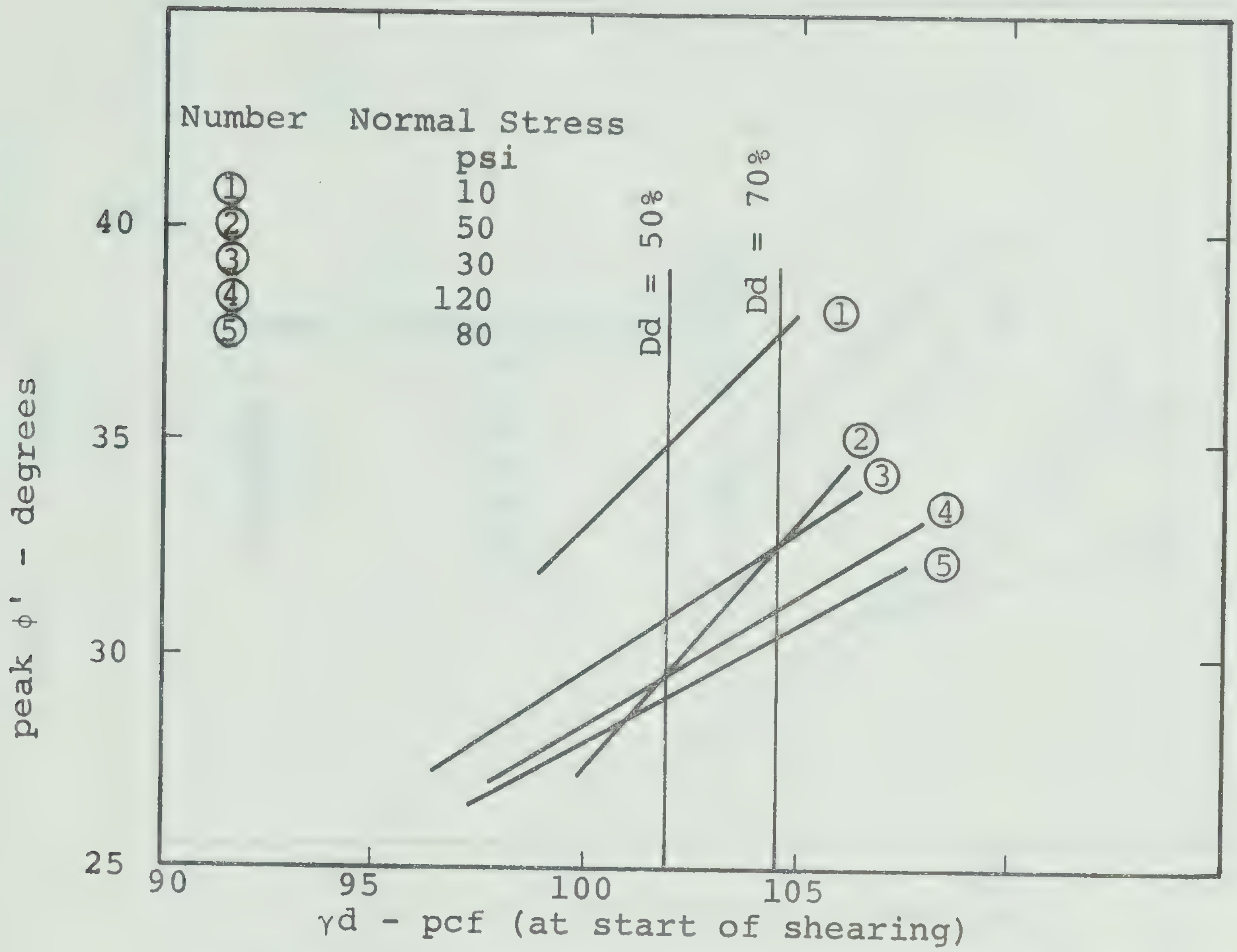


Figure 3.18 Results From Direct Shear Tests
Ottawa Sand (C-109)

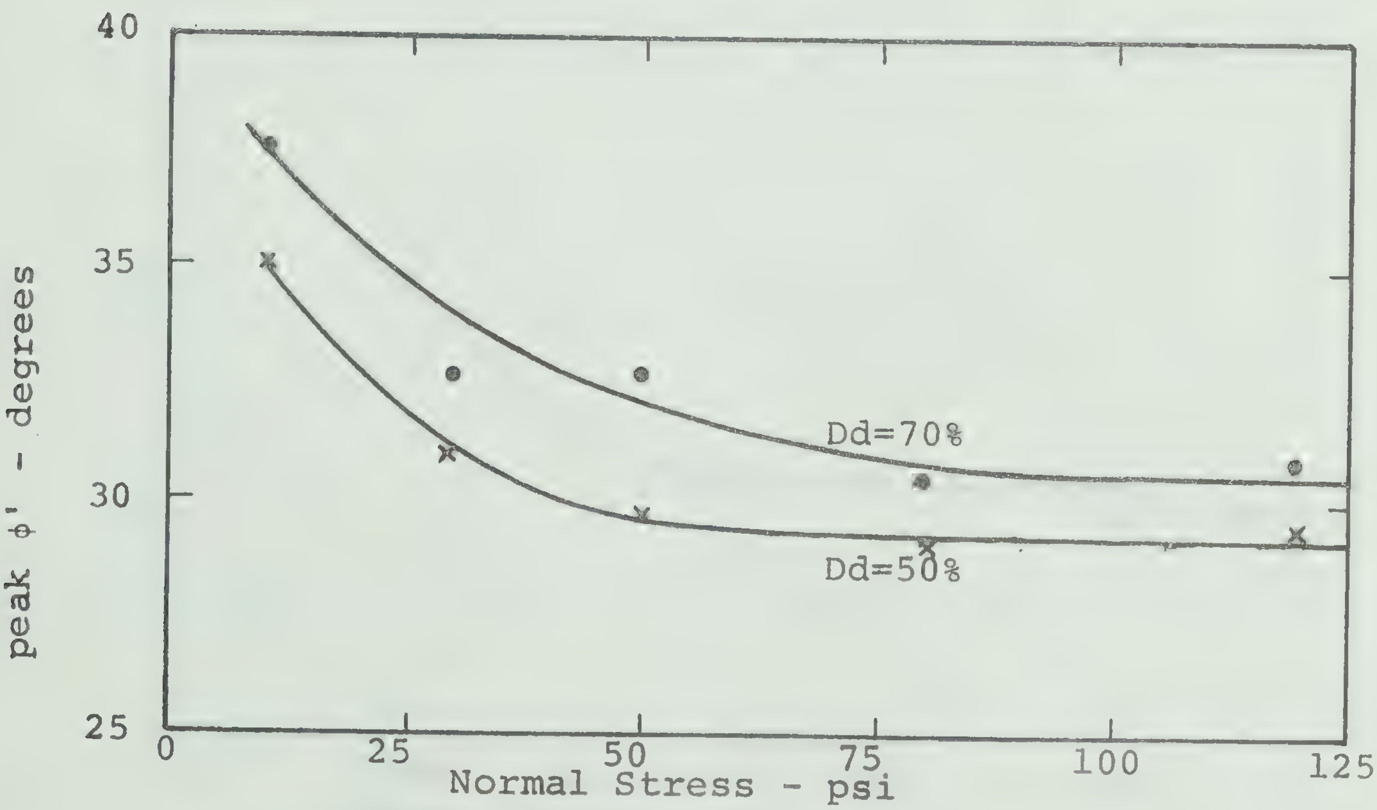


Figure 3.19 Peak φ' vs Normal Stress
(Reference Figure 3.18)

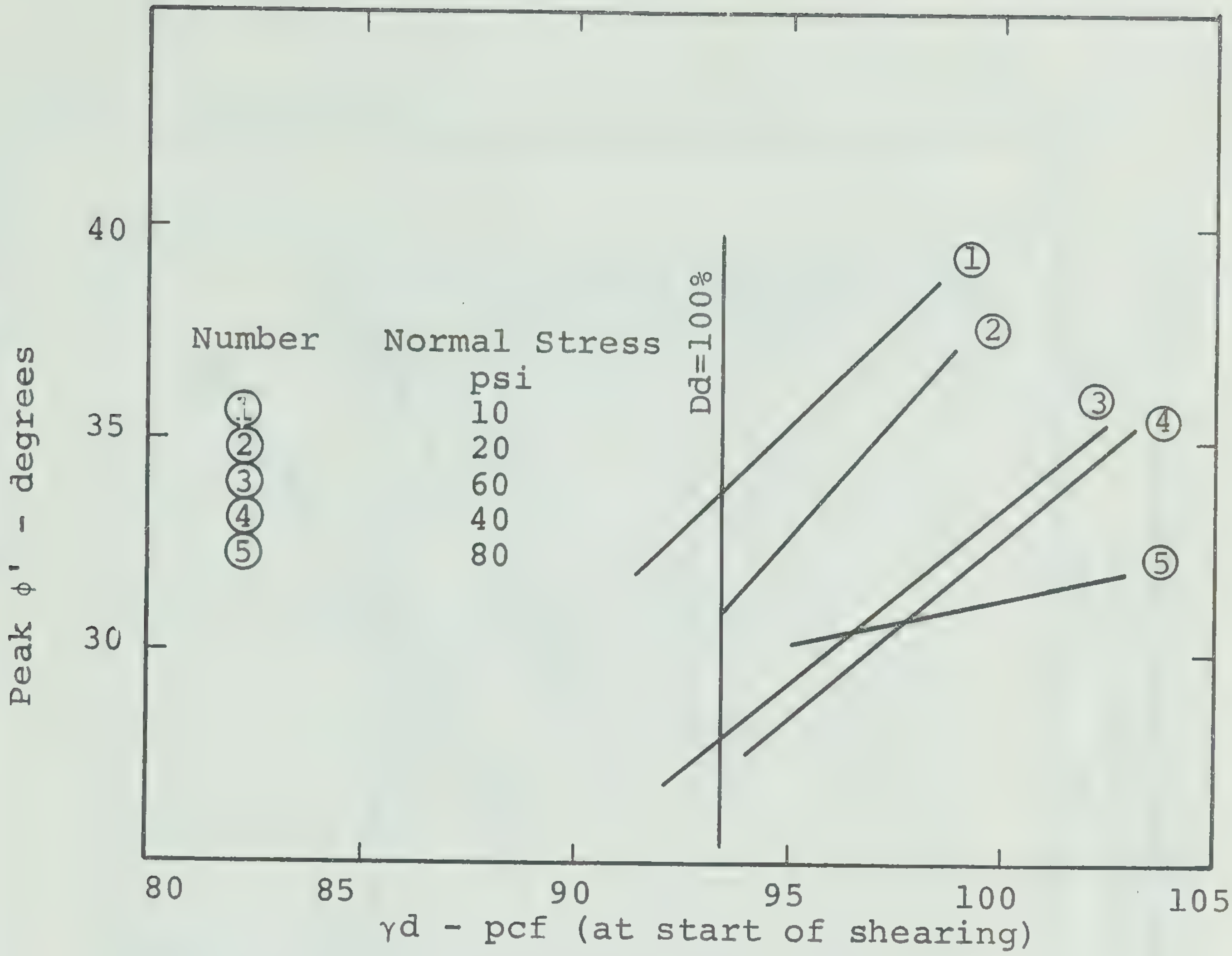


Figure 3.20 Results From Direct Shear Tests GCOS - Sand

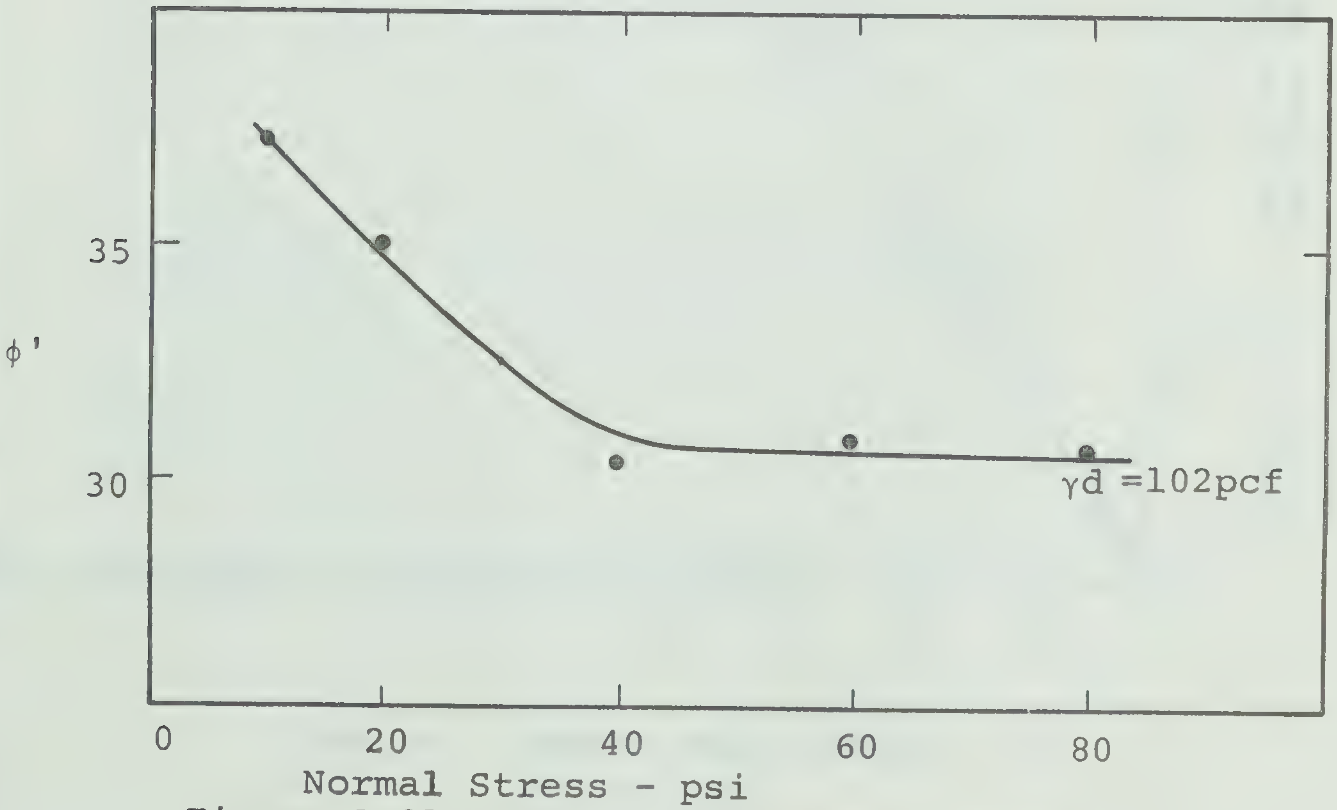


Figure 3.21 Peak ϕ' vs Normal Stress (Reference Figure 3.20)

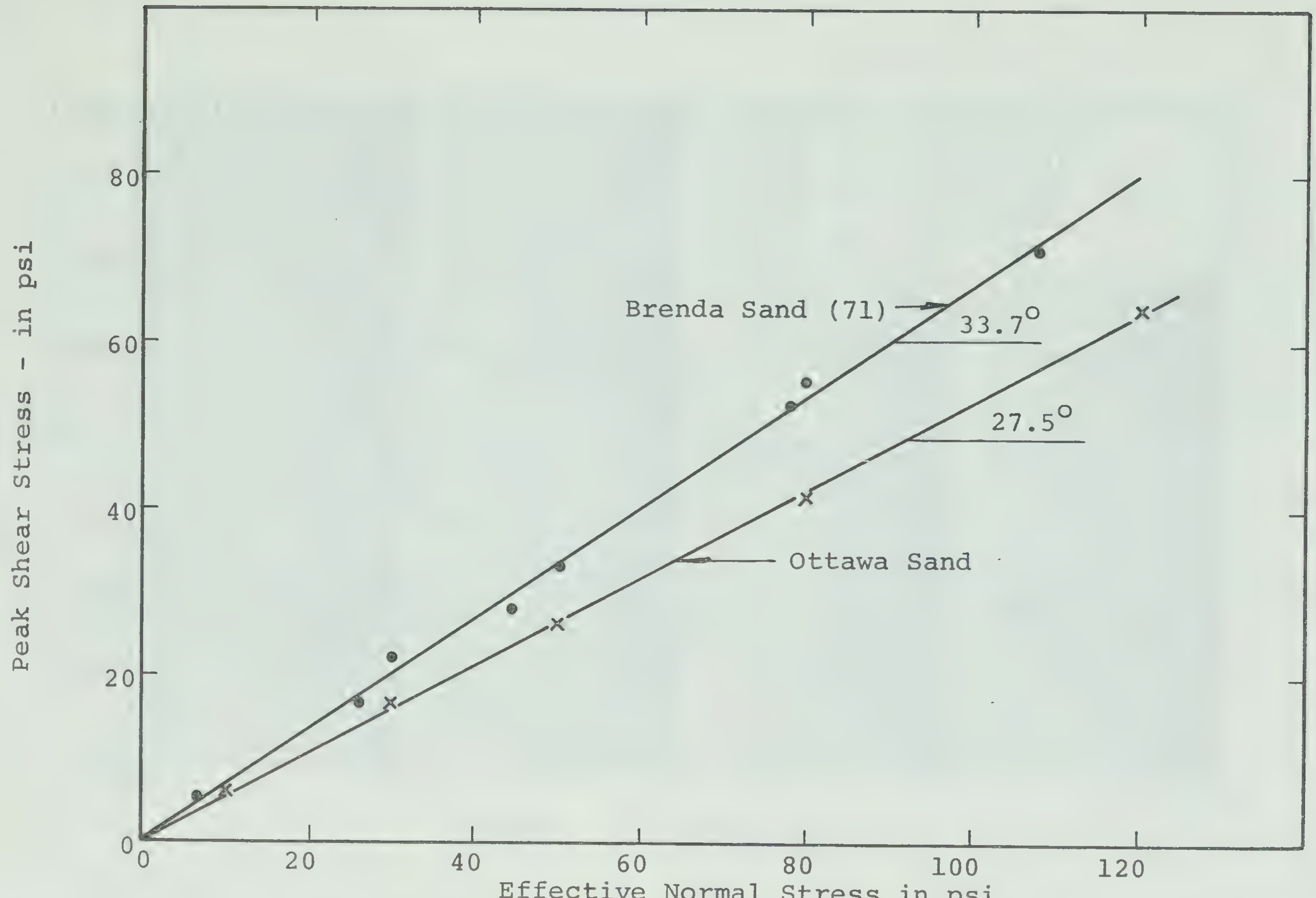


Figure 3.22 Results From Direct Shear Tests for Sands Initially at Zero Relative Density

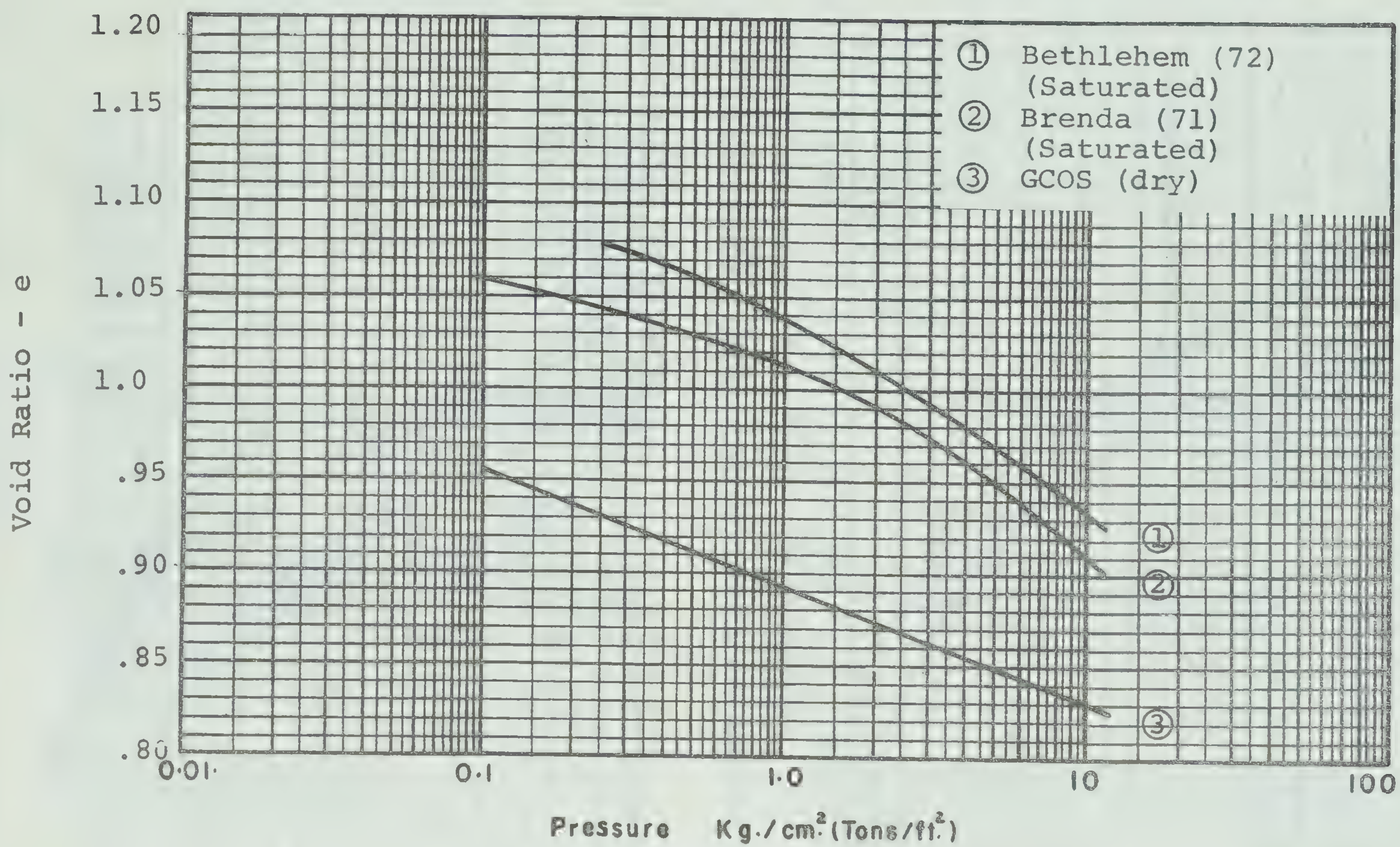


Figure 3.23 Consolidation Curves for Sands

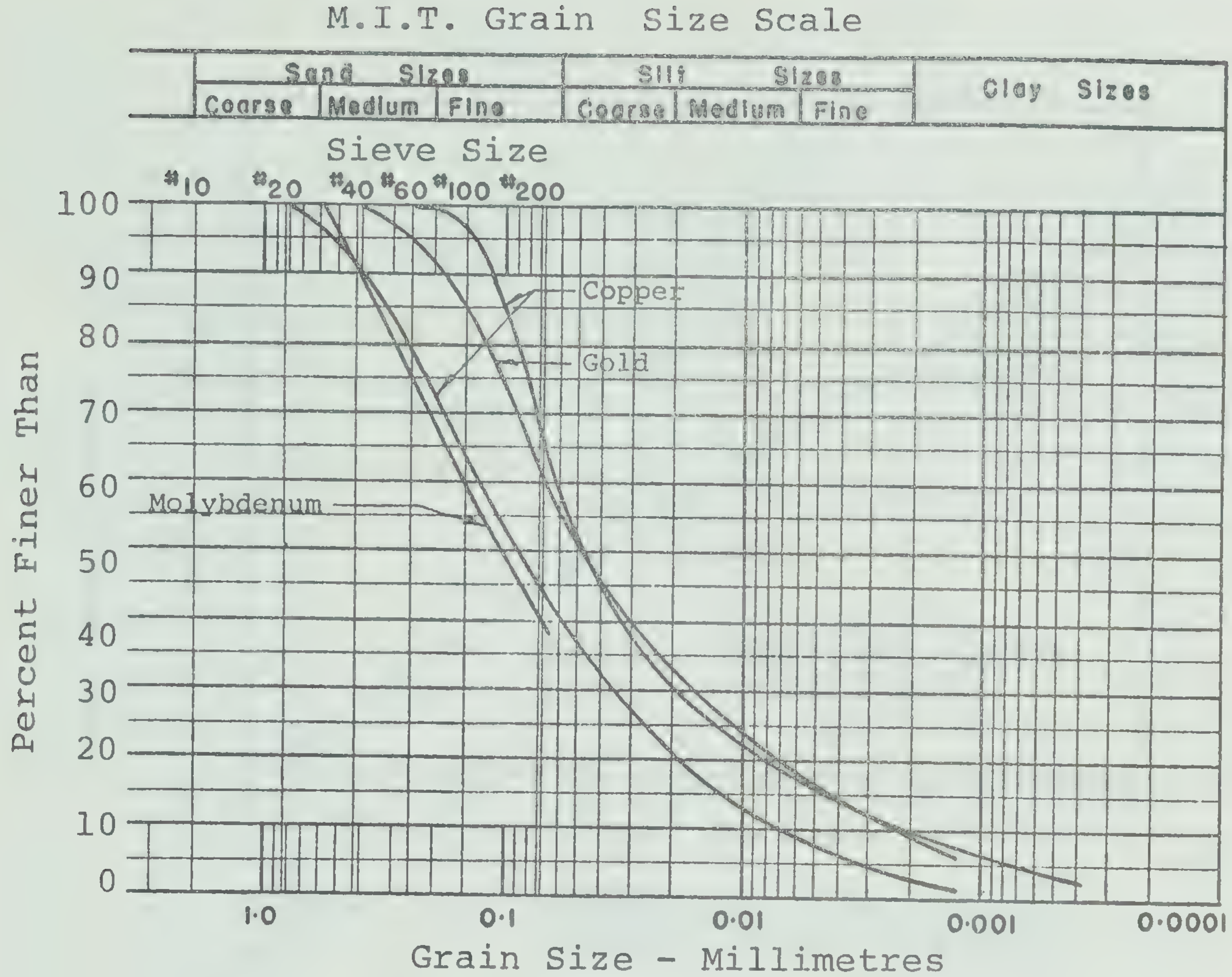


Figure 3.24 Grain Size Curves of Typical Tailings

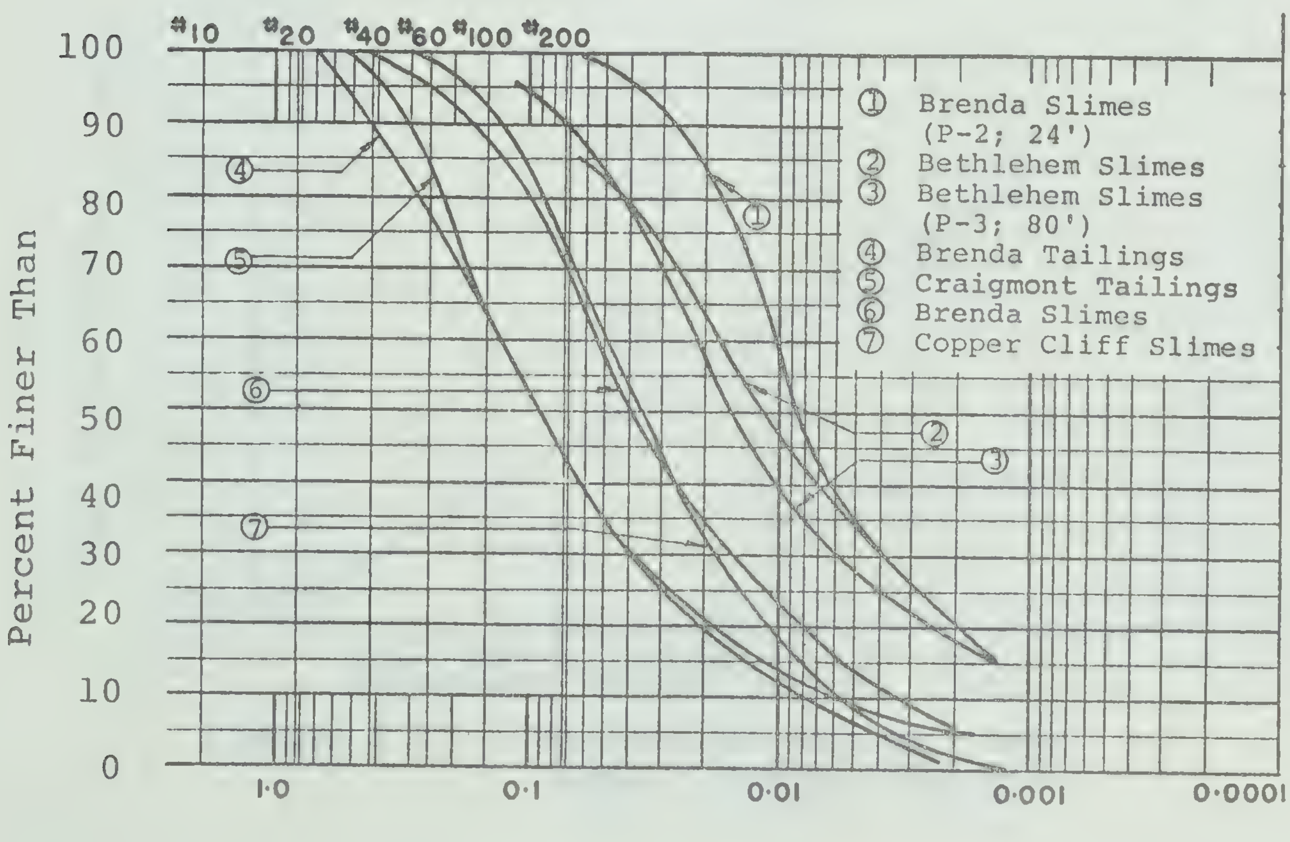


Figure 3.25 Grain Size Curves of Slimes and Tailings Tested in This Study

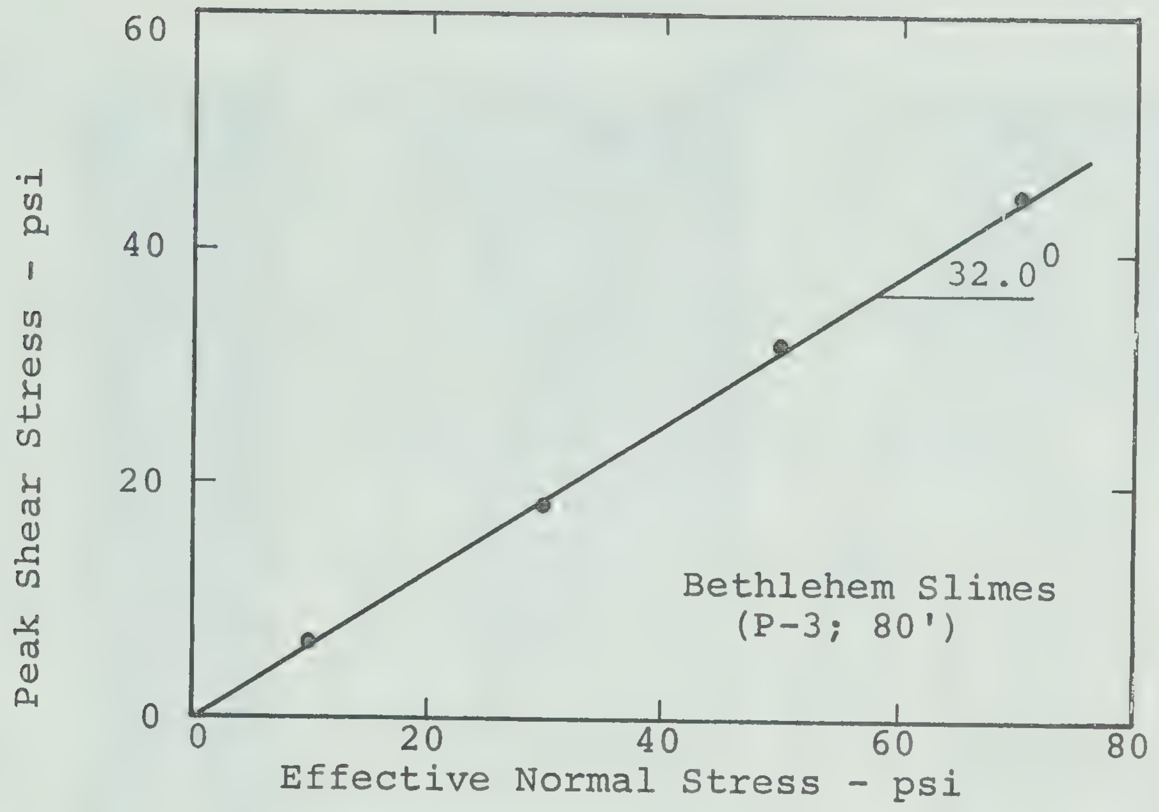


Figure 3.26 Results of Direct Shear Tests - Typical Slimes

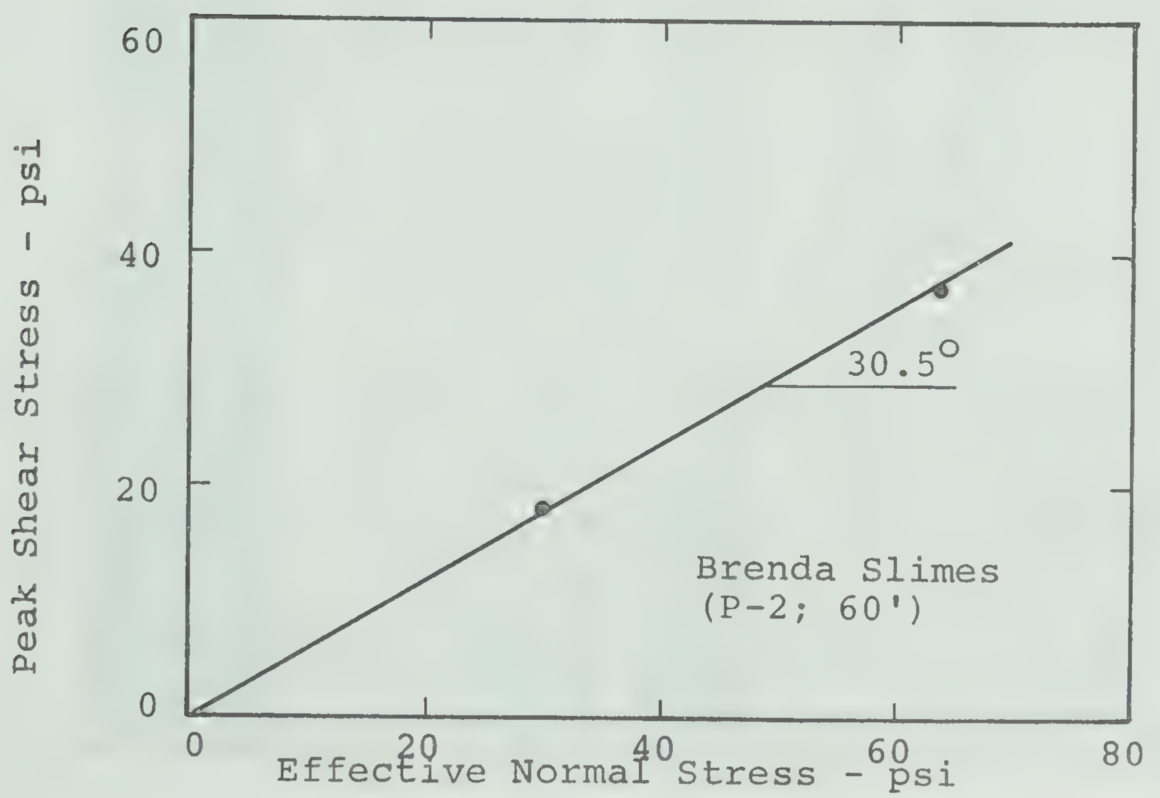


Figure 3.27 Results of Direct Shear Tests - Typical Slimes

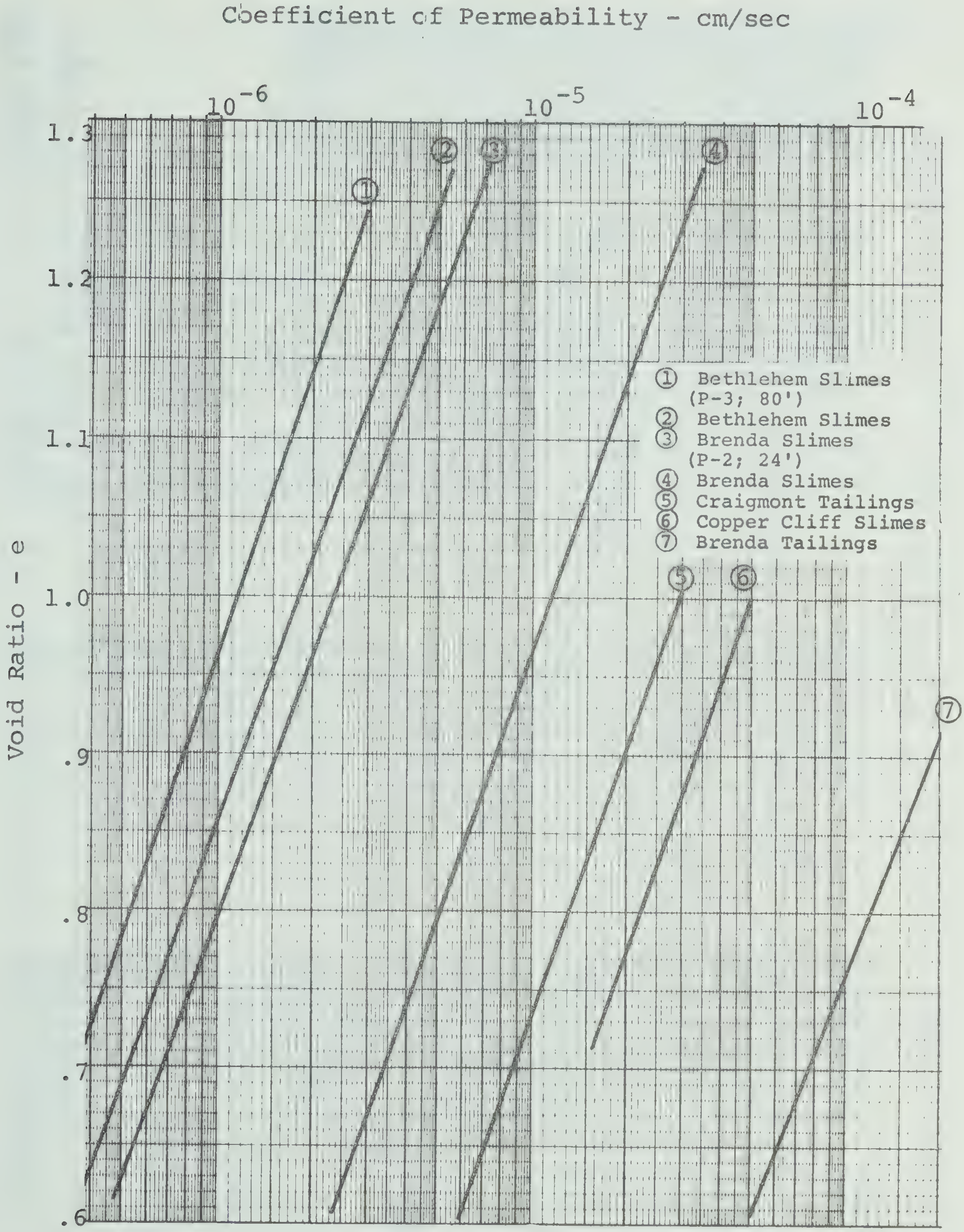


Figure 3.28 Permeability Data For Tailings and Slimes

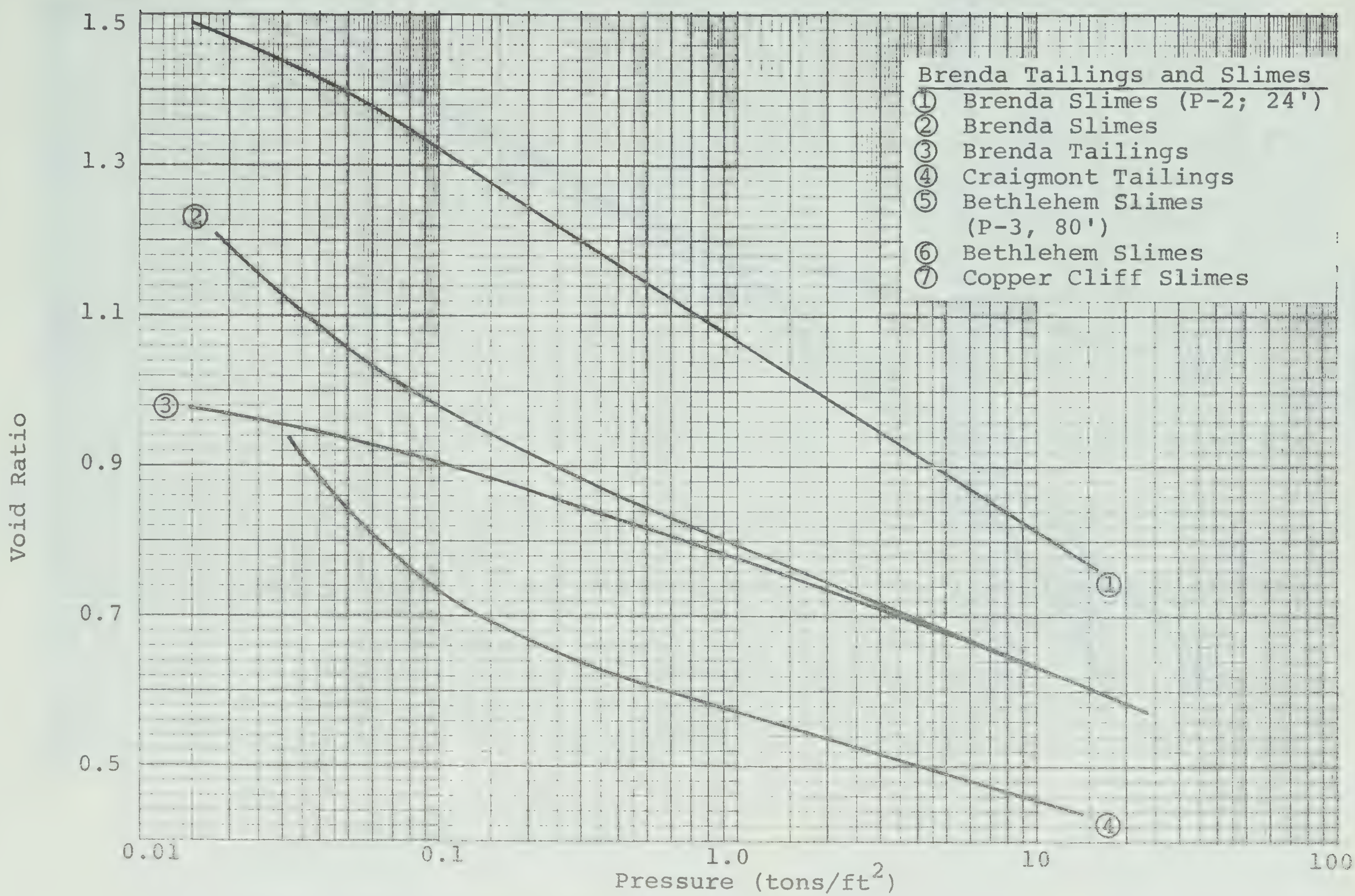


Figure 3.29 Consolidation Data-Tailings and Slimes

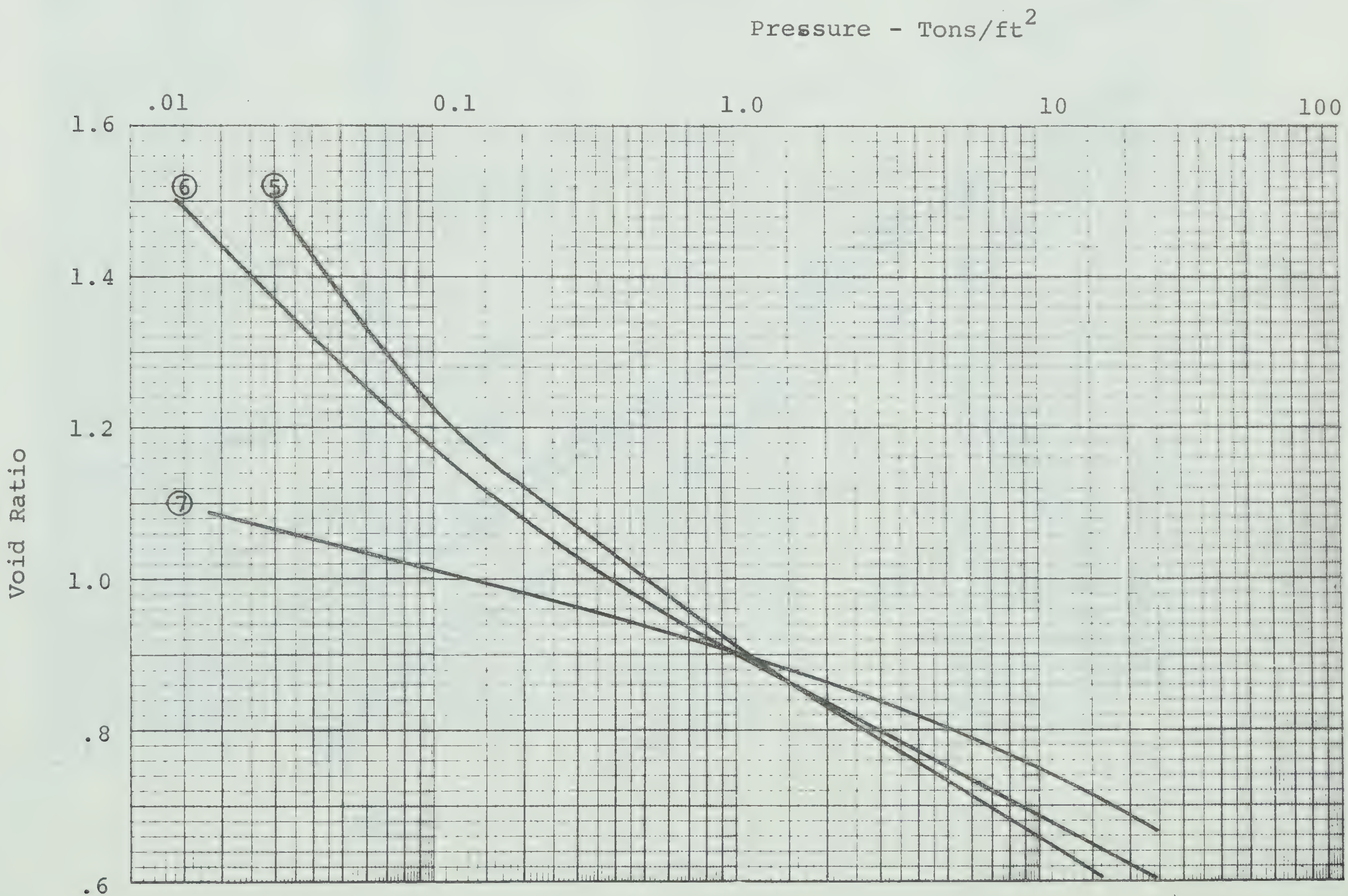


Figure 3.29 (cont'd)

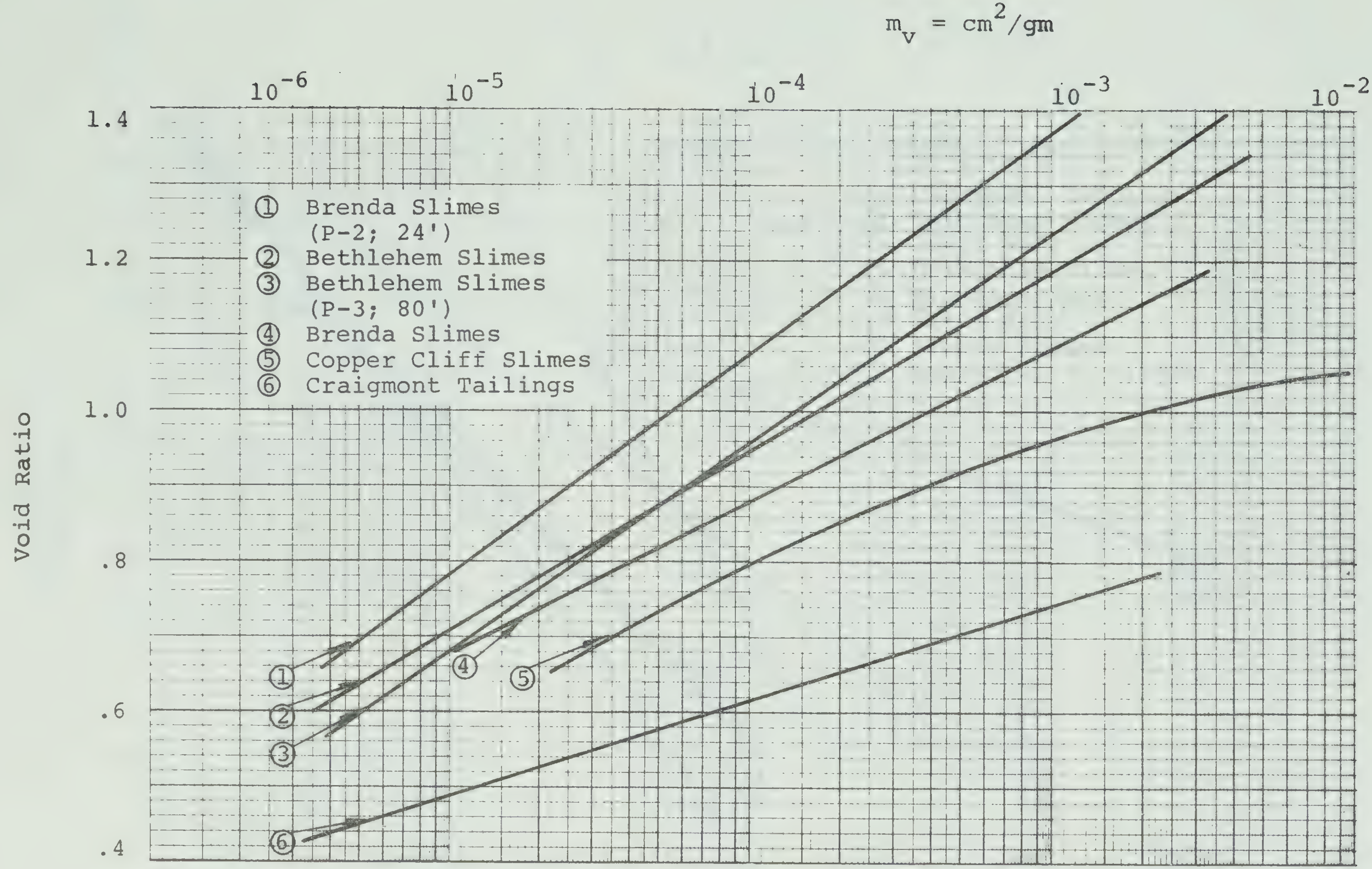


Figure 3.30 Void Ratio vs log m_v-Slimes and Tailings

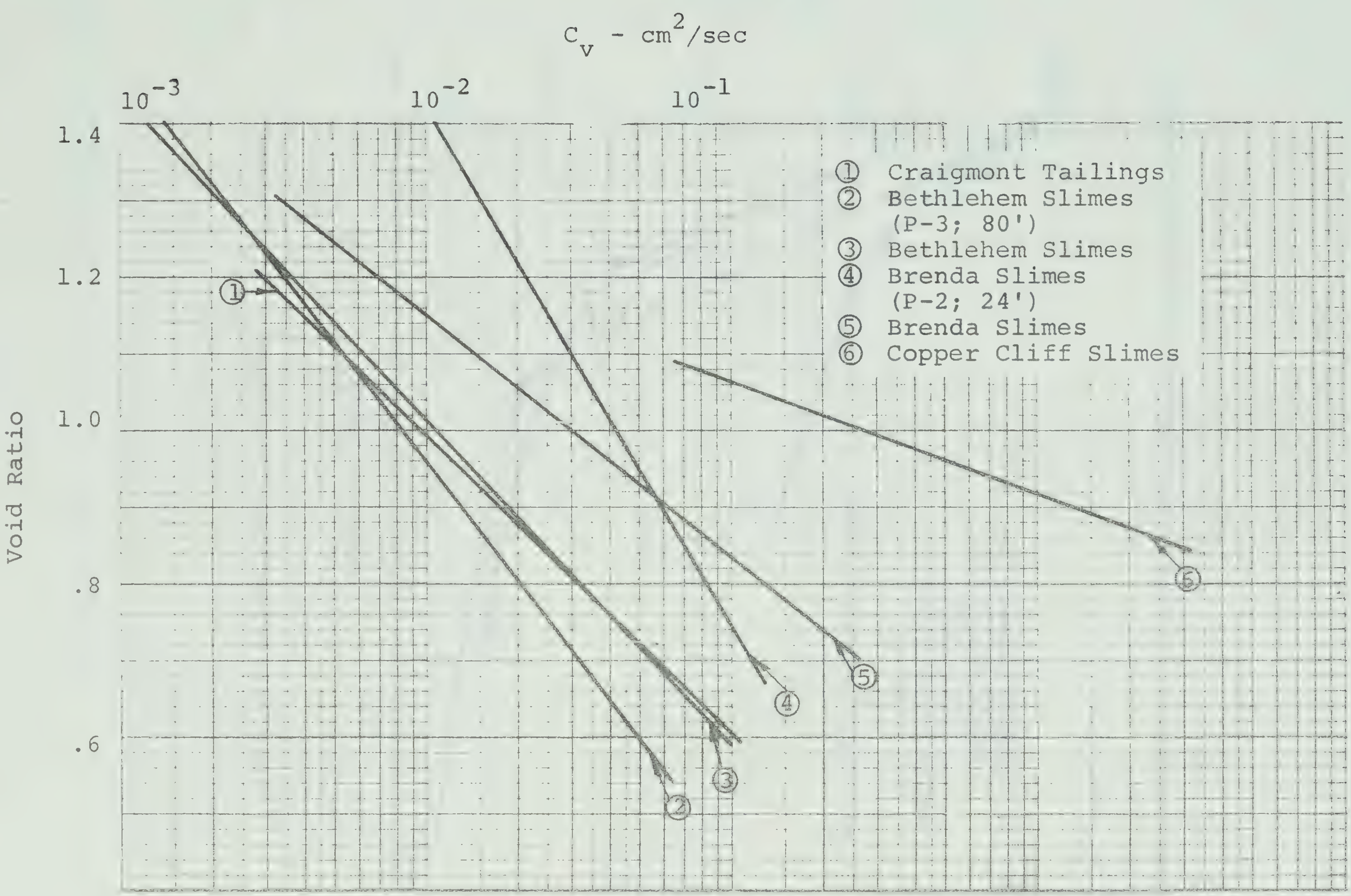


Figure 3.31 Void Ratio vs. Log C_v-Slimes & Tailings

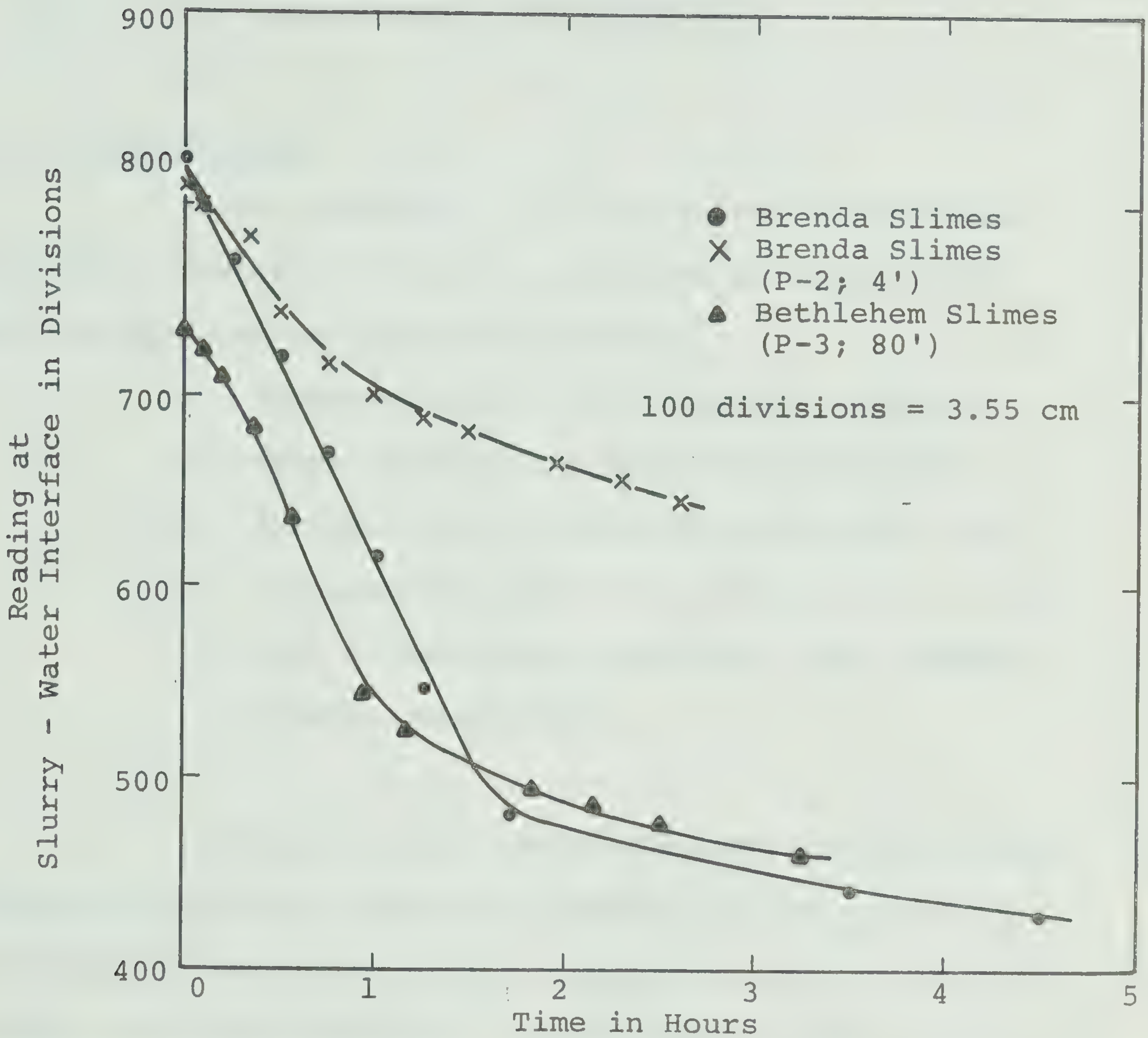


Figure 3.32 Results of Sedimentation Tests

CHAPTER IV

RESULTS AND OBSERVATIONS FROM FIELD INVESTIGATIONS

4.1 Introduction

In an embankment, the major items of material behaviour that are likely to influence its structural performance can be listed as follows:

i) compressibility of embankment material;

ii) shear strength of embankment material;

iii) permeability of embankment material; and

iv) liquefaction potential and/or general behaviour of embankment material under seismic loading conditions.

In hydraulically constructed embankments of the coarse fraction of tailings, compressibility and shear strength of the material are, generally, not an issue but items (iii) and (iv) are. As discussed in Chapter II, for seismic considerations, a minimum relative density of 60 percent is usually recommended in the construction of tailings dams. Thus, from the point of view of monitoring construction of these embankments, measurements of in situ density and permeability are of prime interest.

In this study, existing methods of measuring these parameters in situ have been critically reviewed and new techniques investigated. Also field investigations have been carried out to measure these parameters at three tailings dams being built by different methods. All pertinent details including the test results from field investigations are reported in this chapter.

During the field investigations, samples of slimes and measurements of pore pressures in the tailings ponds were also obtained where possible; the results are included herein. Some general observations relevant to materials handling made during these investigations will be discussed in a later chapter of this thesis.

In fairness to those responsible for the safety of the tailings dams investigated during this research program, it should be pointed out that field investigations were carried out with the sole purpose of obtaining typical information related to the factors discussed above. The investigations were not detailed enough, and neither were they intended to be, to assess the stability of any overall structure or suitability of any significant portion of a disposal system. Although it is the writer's sincere hope that observations and conclusions recorded in this thesis will be of use in improving design and/or construction methods on many existing (including those investigated) and

future tailings dams, it is not his intention to pass judgement on any procedures being followed at the properties visited during this program. It is felt that to do so will require further specific work pertinent to the objectives of the assignment in question.

4.2 Tailings Dams Investigated

The three tailings dams investigated are as follows:

i) Brenda Mines located near Kelowna, B.C.;

ii) Bethlehem Copper Corp. located in the Highland Valley near Ashcroft, B.C.; and

iii) Craigmont Mines located near Merritt, B.C.

Figure 4.1 is a map of British Columbia showing locations of the above mines.

A brief description of the type and size of dam, and the method of construction and materials being used at each site is presented below.

4.2.1 Brenda Tailings Dam

As shown in Figure 4.1, Brenda Mines is located on a mountain plateau west of Okanagan Lake in south central British Colubmia, approximately 40 miles from the city of Kelowna. The mine produces copper and molybdenum concentrates from a low-grade open pit operation at a rated

capacity of 24,000 tons per day. At this rated capacity, the mine was to operate for 20 years producing approximately 175 million tons of tailings for disposal. Recent modifications are, however, underway to increase the daily throughput capacity of the mill to 30,000 tons per day.

The tailings dam is located in a narrow sloping valley. Therefore, a relatively high dam is required to provide the necessary storage capacity. The designed ultimate height of the dam is 450 feet above the stream bed at the downstream toe. Immediately below the crest of the dam, the ultimate height of the embankment above the stream bed will be approximately 370 feet.

Prior to commencing construction of the sand dam in the spring of 1970, a starter dam (130 feet high), a downstream rock dam (almost 200 feet high) and a suitably designed underdrainage system were constructed as shown in Figure 4.2. This figure shows a plan view and a typical section of the dam. Figure 4.3 is a photograph of the tailings disposal concept at Brenda Mines and Figure 4.4 is a photograph of the Brenda tailings dam as existing in June, 1973. All seepage through the dam is collected at a "Reclaim Dam" placed downstream of the main dam and pumped back into the mill system.

From the cyclones placed high on an abutment, the

sand is transported and placed hydraulically. The construction is by the centreline method as described in Chapter II. A further description of the construction details is presented later in this chapter.

The slimes from the cyclones are transported by pipeline to the dam and discharged into the pond by conventional spigotting from along the crest of the dam in an upstream direction. The operation is designed to form a broad, low-permeability beach between the pond and the sand dam.

4.2.2 Bethlehem Tailings Dam

Bethlehem Copper Corporation Limited operates an open pit copper mine located in the mineral rich Highland Valley in south western British Columbia. As shown in Figure 4.1 the mine and the concentrator are situated at an approximate distance of 28 miles by road from Ashcroft, B.C.

Mining operations commenced in December, 1962. The mill has undergone several modifications since the initial construction to steadily increase the throughput capacity. For example, in 1966, the mill operated at a throughput capacity of 11,000 tons per day (tpd), in 1969 at 15,000 tpd and currently, the mill is operating at about 17,000 tpd.

A tailings disposal area has been developed by constructing a dam across a valley located adjacent to the concentrator. The dam is constructed of waste rock from the open pit, with an upstream filter zone of cycloned tailings sand. The rock dam is approximately 5000 feet in length and varies in height to a maximum of 300 feet in the centre of the valley. The rock dam has now been completed to its full height. Pond level as of June 8, 1973, has reached an elevation of within 115 feet of the crest elevation of the rock dam. Tailings sand is being placed upstream of the rock dam from the cyclones located along the dam crest. Sand placement has been virtually completed along approximately 1000 foot sections of the dam at both ends as of June, 1973. All testing on the dam was restricted to these completed or nearly completed areas. Figure 4.5 shows a typical section through the dam at these locations. Figures 4.6 to 4.8 are photographs of the tailings dam and the disposal area.

At the downstream toe of the rock dam, a catchment ditch and storage pond have been constructed to intercept seepage through the dam and also a certain amount of ground water discharge from the springs in the immediate vicinity. As in the case of Brenda dam, provisions have been made to recirculate this intercepted water back into the mill system.

Slimes are discharged into the pond in part directly from the mill and in part, from the overflow of the cyclones on the dam. It is of interest to note that, generally speaking, the slimes slurry at Bethlehem tailings dam tends to pond against the sand, although clear water can only be seen at the back of the pond away from the dam.

It should be noted that the main retaining structure at the Bethlehem site is constructed of waste rock and therefore cannot be classed as a typical tailings dam. This site has been included in the present investigation primarily to study the in situ properties of the large volume of cycloned sand which has been placed against the rock dam.

4.2.3 Craigmont Tailings Dam

The Cragimont property is located in the Interior Plateau region of south western British Columbia. As shown on Figure 4.1 the site is located at an approximate distance of 10 miles by road from Merritt, B.C. The mine is situated on the lower slopes of the Promontory Hills which rise to the west from the Valley of Stumbles Creek.

Mining operations commenced in September 1961. At the present time, the mill operates at a rated throughput capacity of 7500 tpd, with approximately 50 to 60 percent of the mine output coming from the underground

operations and the remainder from stockpiles made during earlier open pit mining.

The tailings dam has been constructed across the Stumbles Creek Valley immediately south of the mine site. The proposed maximum height of the dam is about 100 feet. Maximum height at the time of the field investigations was of the order of 82 feet. Initially, a starter dyke about 15 feet high was constructed across the valley at the downstream toe of the final dam and the subsequent construction of the dam has been by the conventional upstream method as described in Chapter II. Figure 4.9 illustrates a typical section of the dam.

4.3 Brief Description of Sand Placement at the Above Dams

At the three tailings dams investigated during this study, methods of sand placement can be summarized as follows.

i) At Brenda tailings dam, considerable water is added to the sand from the second stage cyclones placed at the abutment to permit transport of the sand through pipelines to the dam. Because of the excess water it has been necessary to place sand by discharging the sand-water mixture in cells (approximately 100 feet x 300 feet) essentially, following the conventional hydraulic fill methods. The cells are constructed by bulldozers pushing up sand dykes around the areas designated for sand placement.

The sand-water slurry is discharged into a cell at one end and the excess water decanted at the other. The hydraulic cell method results in sand being placed in 8 to 10 foot lifts with a sloping surface at about 25 horizontal to 1 vertical (25:1). Figure 4.10 presents photographs illustrating this method of construction.

ii) At Bethlehem tailings dam, sand underflow from the cyclones situated on the crest of the dam is discharged directly into its final resting place upstream of the rock dam. The sand slurry at the time of discharge is at a water content of 45 to 50%. All excess water appears to drain in a downward direction with no free water noticeable at the surface. Sand is being placed at this site by the above method in a continuous operation to produce fill lifts in excess of 20 to 30 feet before the cyclones are moved to a new location. Side slopes in the range of 4:1 to 5:1 are generally achieved.

iii) At Craigmont tailings dam, construction is by the conventional upstream method. The height of the dam is raised by timber forms in 2-foot increments. A portion of the tailings is discharged from along the timber forms through small spigots located at close spacing. The remainder is discharged through relatively large overhead pipes extending across the beach to a distance of approximately 75 to 100 feet from the timber forms.

Construction of the dam is essentially restricted to the summer months. In the construction schedule followed at this site, each 2-foot lift on the dam is allowed to rest for several months before the next lift is placed. During the off season a certain amount of tailings are also discharged from along the perimeter of the pond.

It should be noted at this time that in this method, excess discharge through the overhead pipes can result in high areas at the point of discharge on the beach resulting in backponding and deposition of fine slimes near the crest of the dam. Figures 4.11 and 4.12 illustrate the method of construction being followed at the Craigmont tailings dam.

4.4 In Situ Density Measurements

4.4.1 Methods of Measuring In Situ Density

In construction control, in situ densities can usually be measured by any one of the several methods listed under "ASTM Test for Density of Soil in Place", such as the sand cone, (D1556-68) the rubber balloon (D-2167-72) and the drive cylinder (D-2937-71) methods. However, all these tests are usually restricted to shallow depths. These tests may be performed at depth in test pits and shafts excavated for the purpose but then, they tend to be very time-consuming and costly. In situ density determinations at depth can sometimes be made by securing undisturbed

samples of the sand which also tend to be costly. Furthermore, undisturbed samples of sand are difficult to obtain, particularly below ground water level, to say the least. In recent years nuclear devices have been developed for measurement of in situ densities. The nuclear devices and their use for this purpose are discussed later in this chapter.

At the present time, when assessment of in situ densities at depth is needed, penetration tests are more likely to be used for the purpose. Penetration tests are quite easy to perform and hence are relatively economical. There are, essentially, two types of penetration tests commonly used in earthwork engineering - the dynamic and the static penetration tests. These tests will be discussed below in the order cited.

In North America, the Standard Penetration Test (SPT) is the most widely used test of its kind. A description of this test can be found in Terzaghi and Peck (1967). Besides SPT, several other dynamic penetration tests have been developed. These involve the driving of drill rods equipped with a conical point under the action of a drop hammer in a manner similar to that employed in the Standard Penetration Test. To eliminate side friction on the rods, the conical point is usually larger in diameter than the rods. In most cases, the point is an expendable steel cone.

This type of test is easier to perform than the SPT and provides a continuous log of penetration resistance. However, the test is far less standardized; the test details can vary according to its intended use. Results obtained with one such penetration test are presented by Klohn and Maartman (1973).

Static penetration tests have been quite extensively used in Europe for many decades. At the present time, Dutch Cone apparatus appears to be the one most widely used. Sanglerat (1972) has provided a most comprehensive discussion on the Dutch Cone penetrometer, its uses and a summary of experiences with this equipment from all parts of the world.

In the static penetration test, a cone is pushed into the ground at a constant rate under continuous static pressure. In the Dutch Cone penetrometers in current use, the point resistance and side friction on the rods can be measured separately. The point resistance is usually taken as a measure of in situ density.

4.4.2 Usefulness of Standard Penetration Test in Measurement of In Situ Relative Density

In conjunction with sampling of sand in exploratory drilling, the Standard Penetration Test provides a measure of penetration resistance which can be of great value in an assessment of relative denseness of the material. The

penetration resistance can be used to relate soil conditions at a given site to the experiences gained at other sites, as Terzaghi and Peck (1967) appear to have done with respect to foundation performance. The usefulness of Standard Penetration Test in a quantitative determination of in situ relative density, however, is rather questionable as will be shown below.

It has been recognized for some time that penetration resistance is not only a function of in situ density but also the overburden pressure acting on the material. Several investigators have produced correlations between penetration resistance (N-values), relative density and overburden pressure. Figure 4.13 presents the most frequently used results of Gibbs and Holtz (1957) on the subject. Peck and Bazaraa (1969) indicate that the above correlations of Gibbs and Holtz, generally overestimate in situ relative density of a deposit as shown by Bazaraa (1967) whose results can be expressed by

$$N = 20\, D_d^2 (1 + 2P)\,, \quad \text{for } P \leq 1.5 \text{ Kips per sq ft}$$

$$\ldots\ldots(4.1)$$

$$N = 20\, D_d^2 (3.25 + 0.5P), \quad \text{for } P \geq 1.5 \text{ Kips per sq ft}$$

where N = the Standard Penetration blow count in blows per foot,

D_d = relative density (expressed as ratio),

and

P = overburden stress in Kips per sq ft.

In a case study, Lacroix and Horn (1973) find that the results of Gibbs and Holtz correlate quite well with those of a natural deposit but not with those from a compacted fill; results from the compacted fill correlate with Bazaraa's work, however. Lacroix and Horn indicate that perhaps the reason for this discrepancy is that increases in the values of N are not essentially due to increases in the vertical effective stress, but are, rather, the result of an increase in the horizontal effective stress. Schmertmann (1970A) notes that surface compaction can improve the soil in two ways, i.e., it increases both density and lateral stresses. Such lateral stress increase is likely to increase both the N-values and the cone bearing resistance as indicated below,

> "Before compaction, a cone bearing of 25 tons per sq ft matched with a relative density of 44%. After compaction the cone bearing jumped to 56 tons per sq ft for essentially the same relative density of 47%. A cone bearing of 56 tons per sq ft in this sand when uncompacted would ordinarily suggest a relative density of 57%. The change is most likely due to increased lateral stresses."

Results from two test locations at Brenda tailings dam where Standard Penetration Tests were performed <u>do not correlate</u> with the work of either Gibbs and Holtz or Bazaraa. This will be further discussed later in this

chapter.

Recently Lacroix and Horn (1973) have summarized the results of various investigators; the results are presented in Figure 4.14. A review of this figure indicates that there is a large range of values of relative density that can be interpreted for a given N-value from the results of these investigators. It should be noted at this point, however, that it is more than likely that the above investigators had used different test procedures to determine maximum and minimum densities. Therefore, it is entirely possible that significant differences in their correlations (Figure 4.14) could be simply the result of differences in the computed values of relative density for a given in situ density.

In conclusion, on the basis of the above discussions, it can be said that the N-value in a Standard Penetration Test depends upon:

i) density of sand,

ii) overburden stress, and

iii) coefficient of earth pressure at rest (K_o) which depends on the stress history of the deposit (such as compaction or overconsolidation).

Furthermore there is evidence that penetration resistance also depends upon such variables as angularity of grains (Holubec and D'Appolonia, 1973) and perhaps grain size distribution. Also significant differences in N-values can occur just due to the crude nature of the test from one site to another or at different times at a single site. On the basis of the above considerations it can be safely concluded that the SPT is too crude and approximate a test for a global correlation, such as that of Gibbs & Holtz, to be of any practical value in the quantitative assessment of in situ relative densities. This is particularly true in the case of tailings dams where relative densities are likely to vary over a rather narrow range.

4.4.3 Usefulness of Static Cone Penetration Test In Measurement of In Situ Relative Density

As indicated by Schmertmann (1970A) the static cone penetration test is far superior to the rather crude SPT. The penetration resistance can be measured precisely and accurately. The test is relatively easy to perform and produces a continuous penetration log for full depth of a test hole. The value of this test in design of foundations where settlement rather than bearing capacity is the governing criterion, has been clearly demonstrated by Schmertmann (1970B).

Usefulness of the cone pentration test for a quantitative assessment of in situ relative density, never-

theless, remains questionable. Once an appropriate correlation is established on a given site or in a local area, the cone test can be effectively used to assess in situ density but the problem lies with obtaining appropriate correlations. Some investigators (Schmertmann, 1970A) appear to have established fairly reliable correlations between cone penetration resistance and relative density through extensive use in a local area. Such correlations, however, cannot be assumed to apply to deposits in other areas with different histories. In fact, any attempts to establish global correlations empirically will be subject to the same objections as those ascribed to similar correlations for the Standard Penetration Test.

Consideration was given to obtaining the necessary correlations theoretically by invoking the theory of bearing failure which is controlled by the shear strength or angle of shearing resistance (ϕ') of the sand, all other things being equal. As can be estimated from Figures 3.16 and 3.17 in Chapter III, the ϕ' for a typical tailings sand varies over a narrow range of no more than 5 degrees for the relative densities likely to be encountered in tailings dams. In addition to ϕ', the penetration resistance in a static cone test is influenced by most of the variables previously ascribed to the Standard Penetration Test. In view of these variables, the small change of 5 degrees in ϕ' is not likely to prove to be a sufficiently sensitive

indicator for a meaningful theoretical correlation between penetration resistance and in situ relative density.

4.4.4 Nuclear Devices and Their Use in Measurement of In Situ Density

4.4.4.1 General

These are non-destructive testing devices that use the neutron-energy-absorption or gamma-ray scattering techniques to measure the moisture content or density of rock and soil materials. These nuclear devices have been in the process of development for about 20 years. The equipment first became available commercially in the middle 50s. At the present time, surface and subsurface-type moisture and density probes are manufactured and used in many countries.

The surface type probes are now frequently used by large governmental agencies for compaction control on projects involving large volumes of fill such as roadways, airports, etc. The subsurface-type probes appear to be mainly used in research at the universities, in ground water surveys by hydrologists and in exploration by the petroleum industry. Their use in geotechnical engineering has been, however, rather limited, to say the least. For example, the recent ASTM symposium on "Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils" contained only one paper where nuclear devices had been used to measure in situ densities and then only at

shallow depths.

In view of the limitations of the conventional penetration tests as described in the previous sections, subsurface-type moisture and density probes were investigated for use in this study. The results are very encouraging. Pertinent details relevant to the subsurface probes used in this investigation are presented below.

4.4.4.2 Brief Description of Theory

The basis for nuclear determination of water content and density of soil has been described by Waananen (1964) as follows:

> Water Content - "High energy (fast) neutrons emitted into soil by an appropriate radioactive source will scatter in a random manner and lose energy in elastic collisions with low atomic-weight nuclei. Neutrons are particles of matter having a mass almost equal to that of a hydrogen atom, the one atom in most soils with a comparable mass; thus, the scattered neutrons lose more energy in collisions with the hydrogen atom than with any other atom generally found in the soil. Some of the neutrons slowed by the collisions return to the vicinity of the source and can be counted by a thermal (low-energy or slow) neutron counter. The count obtained is an indication of the number of hydrogen atoms present. Water is the principal source of hydrogen atoms in soils and the count provides a measure of the amount of water in the soil surrounding the source of "fast neutrons".
>
> Density - "Gamma rays emitted into soil by a gamma source are scattered by the electrons in the soil and lose energy in the process. The number of scattered rays returned to a detector near the source and counted, depends on the average length of the path of the ray to the detector and source, a calibration relationship can be determined that would be effective

> for most soils. The electron density increases proportionally with the density of the soil and causes greater scattering and energy loss. Thus, with increased density, the chances that scattered gamma rays will return to the detector with sufficient energy to be counted become smaller and count rate drops. In common types of soils, therefore, a low gamma ray count indicates high density, and a high count indicates low density."

4.4.4.3 Nuclear Equipment Used

The d/M - Gauge system, manufactured by Nuclear-Chicago Corporation of Des Plaines, Illinois, was used in this study. The system includes a portable scaler 2800A, usable interchangeably with each of the two subsurface probes - depth-moisture probe P-19 and depth-density probe P-20. Standard radiation sources are: moisture, 4-5 millicurie (mc) radium-beryllium, 1,620-yr half-life; density, 3mc cesium 137, 35-yr half-life. The probes are 1.50 inches in diameter. Details of the design and application of the d/M Gauge system have been presented by Neville and Van Zelst (1960). Figure 4.15 is a photograph of this equipment being used in the field. As a check on the performance of a probe with time, a radiation count can usually be taken in the standard shield provided with the probe.

Although properly designed nuclear probes have been used in uncased open drill holes (Preiss and Lahat, 1972), generally speaking, an access tube is needed for advancing a probe to the depth required. In this study, a 20 gauge steel (wall thickness of .0375 inches) tube with

inside diameter of 1.55 inches was used for this purpose.

4.4.4.4 <u>Calibration of Nuclear Probes</u>

Calibration of the probes was carried out in bulk samples of tailings materials placed in a large container suitably designed for the purpose. The calibration container was approximately 20 inches in diameter and 30 inches in height. The size of the container was chosen after tests had shown that in this size container, the boundary effects were eliminated. It was provided with a screen base about 2 inches above the bottom. A drain was provided near the bottom of the container below the level of the screen base.

The probes were calibrated by placing tailings sands around an access tube placed at the centre of the above container. A schematic sketch of the calibration set up is shown in Appendix B. The material was placed under water until the container was nearly full. Excess water was removed to maintain water level in the container at the surface of the sample. The procedure followed after this was as given below:

i) All necessary weight and volume measurements were taken to determine the initial total (wet) density. Assuming a degree of saturation of 100% and knowing the specific gravity of the material, it was possible at this point to estimate the average initial water content.

But the value of initial water content used for calibration purposes was actually back-figured from the water content measurements at the end of the run as described later in this sequence.

ii) The probes were lowered inside the access tube, one at a time. Duplicate counts of one to two minute duration were taken at various depths within the middle 12-inch depth of the access tube. The average of these readings, and the density-moisture measurements from step (i) provided one point on each of the two sets of curves in Figures 4.16.

To account for changes in the standard count of a probe either due to deterioration of the radioactive source or due to some other random variations, ratios of the count in the soil to that in the standard shield were used for calibration purposes as shown in the Figure 4.16. During this study standard counts were frequently taken to ensure high quality calibrations.

iii) At this point, water was drained from the sample through the bottom tap. Vacuum was applied as required until a reasonably uniform water content distribution was achieved as

indicated by measurements taken with the moisture probe.

iv) The new weight and volume measurements of the sample were recorded and step (ii) repeated.

v) Further drainage from the sample was carried out and steps (iii) and (iv) repeated.

vi) Step (v) was repeated as many times as possible until further drainage was impossible under the maximum vacuum available. At this point, the sample was disassembled and specimens recovered from various depths for water content determinations. The average of these values was considered to represent the water content for the last calibration point. Water contents at the initial and the intermediate stages of the calibration sequence were back-figured from this value.

vii) Steps (i) to (vi) were repeated at various initial densities as required to achieve calibration curves as shown in Figure 4.16.

It should be noted that the calibration curve for the Craigmont tailings in Figure 4.16A is different from that of all other materials. This results from the unusually high iron content in the Craigmont tailings as indicated by a specific gravity of 3.0 (Table 3.3).

Pertinent discussion of the effect of iron on water content determinations by nuclear methods can be found in Burn (1966).

4.4.4.5 Development of a Suitable Testing Technique

In the fall of 1972, a preliminary testing program was undertaken to assess the suitability of the nuclear equipment under actual field conditions. Although some tests were performed at all three dam sites described in this chapter, the largest number of tests were performed at the Brenda tailings dam. Therefore results from Brenda Mines only are presented herein. Tests were performed at four different locations along the crest of the dam, spread over a distance of about 1000 feet.

A tamp and auger procedure was used in the installation of the access tubes (Figure 4.17). Briefly, this procedure consisted of initially pushing an access tube of known length to the bottom of a preaugered hole. The depth of the hole was predetermined so that the access tube had an initial stick-up of 4 feet above the ground surface. After this initial step, the access tube was pushed or tamped lightly a few inches at a time. Between tampings the material from inside the access tube was removed with the aid of an auger. In this manner an access tube could be installed essentially as an open end tube with a minimum of disturbance to the surrounding

material. Readings were taken with the nuclear probes at frequent intervals within the bottom 4 feet of the tube. By selecting various tube lengths, probe readings were taken to a maximum depth of 14 feet in this phase of the investigation.

The results are presented in Table 4.1. The range of values of 82.0 to 89.5 pcf for dry density is well within the range obtained during other investigations on this site (Ripley, Klohn and Leonoff, 1973) using conventional methods. At a few locations, density measurements were also made at shallow depths with the aid of a conventional densometer. The results compare quite well with those obtained by nuclear methods as shown in Table 4.1.

Since testing could only be carried out to limited depths using the above tamp and auger procedure, a different technique of installing access tubes was adopted. The new procedure consisted of driving access tubes closed-ended by means of a drop hammer. The driving mechanism consisted of a light weight tripod fitted with small power winch to operate an 80 pound drop hammer. A 10-foot section of access tube was provided with an end plug having a conical point. Because of the thin wall nature of the access tube, driving was performed on E-size drilling rods resting on the end plug inside the tube. In this manner, the driving force was applied directly to the lower end of the tube and

it was subjected to tensile stresses only in the process.

After the initial 10-foot length of access tube was driven into the ground, 5-foot extensions of suitably modified Ex-Flush-Coupled drill casing were added as required to achieve further penetration. After each 10-foot penetration, density and moisture readings were taken with the nuclear probes within the bottom 10-foot section of access tube. All pertinent equipment details are given in Appendix B.

It was a matter of concern that the above driving technique, although successful in installing the access tube to greater depths than those possible with the tamp and auger procedure, would cause excess disturbance to the surrounding soil. Regarding the net effect on the soil, the driving of the access tube is essentially analogous to the expansion of a cavity in a soil mass. Vesic (1972) has presented a rather detailed treatment of the subject of expansion of cavities. The case of an expanding cylindrical cavity is of particular interest here.

Consider the problem of a cylindrical cavity of initial radius R_i expanded by a uniform distributed radial pressure p. If pressure p is increased, a cylindrical zone around the cavity will pass into a state of plastic equilibrium after a threshold valve is reached. The plastic

annulus will expand until the pressure reaches an ultimate value p_u. At this moment the cavity will have a radius R_u and the plastic annulus a radius R_p. Beyond the plastic annulus, the soil mass remains in a state of elastic equilibrium.

The above problem has been treated by Vesic (1972) based on the following assumptions:

i) the soil in the plastic zone behaves as a compressible solid, defined by Mohr-Coulomb failure criterion and an average volumetric strain Δ,

ii) beyond the plastic annulus, the soil behaves as a linearly deformable, isotropic solid defined by a modulus of deformation, E, and a Poisson's ratio μ,

iii) prior to the application of load, the entire soil mass has an isotropic effective stress q and the body forces within the plastic zone are negligible when compared with existing and applied stresses.

The following equations form the basis for the above work by Vesic:

i) equilibrium equation for radial plane strain,

$$\frac{\partial \sigma'_r}{\partial r} + \frac{\sigma'_r - \sigma'_\theta}{r} = 0 \qquad \text{.....(4.2)}$$

where σ'_r = effective normal stress in radial direction at radius r,

and σ'_θ = effective circumferential normal stress;

ii) equation of Mohr-Coulomb failure criterion for cohesionless sands,

$$\frac{\sigma'_\theta}{\sigma'_r} = \frac{1 - \text{Sin } \phi'}{1 + \text{Sin } \phi'} \qquad \text{.....(4.3)}$$

iii) a relationship stating that the change in volume of the cavity is equal to the change in volume of the plastic annulus plus the change in volume in the elastic zone beyond the plastic annulus.

From the above work of Vesic (1972), the values of p_u, R_p and Δ can be determined. In the present study analyses have been performed for Brenda sand at two typical depths (2 and 20 feet). For a ϕ' of 35^o and from the consolidation data for Brenda sand in Figure 3.23, the values of various input parameters have been computed as given below.

At a depth of 2 feet,

q = .062 tons per square foot

E = 9.0 tons per square foot

μ = 0.3

and at a depth of 20 feet,

$q = .62$ tons per square foot

$E = 90.0$ tons per square foot

$\mu = 0.3$

At both of these depths the values of R_p and Δ are computed so that,

$R_p = 5.5$ inches,

and $\Delta = 1.2$ percent.

For a typical access tube (radius = .81 inches), the volume change of the cavity is 2.06 cubic inches per lineal inch. In the above analysis, the volume change in the elastic zone is 0.95 cubic inches. Hence, the volume change in the plastic annulus is 1.11 cubic inches, equivalent to an average volumetric strain of 1.2 percent.

It is of interest to note that in the above analysis, the values of R_p and Δ are functions of E/q. Since this ratio is constant for the two cases considered above, similar values of R_p and Δ are obtained.

The radius of influence for radiation count with the density probe is a function of the density of the medium in question. In an extremely dense medium such as the lead shield, the radius of influence appears to be less than 2 inches and in loose soils a maximum of 10

inches (Rix, 1965). For the range of densities of interest in the tailings dams, however, it can be assumed that the volume of influence of nuclear radiation is contained within a 6-inch radius or within the plastic annulus of the Vesic analysis. Hence, it appears that the density measurements by nuclear probes are likely to be in error by about 1 percent due to the disturbance caused by driving of an access tube. It should be noted at this point, that Vesic's analysis is usable only in the case of loose to medium dense sand where volumetric strains cause a decrease in volume.

A limited number of tests were performed in the laboratory to assess the net effect of driving an access tube on the measured value of density by nuclear probes. Bulk samples at various densities were prepared of Brenda sand in a manner similar to that employed for calibration purposes in Section 4.4.4.4. Weight and volume measurements were made to determine the initial average density in each case. An access tube was then driven in the centre of the sample in a manner similar to that proposed for the field testing. The results are presented in Table 4.2. It can be seen, in the range of densities tested, that the measured values by nuclear methods are within 1.3 pcf of the initial average densities. Furthermore, it should be noted that under the prevailing low confining pressures, a test at the low initial density of 84.7 pcf shows an increase in density on driving of access tube and that at

the initial density of 87.2 pcf, a decrease in density. Comparative tests were performed at the Brenda and Bethlehem dams using the two techniques of installing access tubes described previously. The results are presented in Figure 4.18. It is encouraging to note that the differences in measured dry densities are comparable to those measured in the laboratory and well within the range of values that can be expected in duplicate tests as shown in Table 4.1.

Tests were also performed at the Brenda tailings dam to compare density results from driven access tubes with those obtained by the conventional drive cylinder method in backhoe test pits excavated for the purpose. The drive cylinder method used was essentially similar to "ASTM Test for Density of Soil in Place by the Drive Cylinder Method (D-2937-71). This type of test procedure was followed because previous testing on this site had indicated that more consistent results could be obtained by this method than those by the conventional densometer (Maartman, 1973). Testing was carried out at two sites situated approximately 500 feet apart along the crest of the dam. Two to three tests were performed with the drive cylinder method at each selected depth to a maximum depth of 17½ feet. All tests below 9 feet were performed from inside of a 4-foot diameter protective casing. The test results are presented in Figure 4.18. A 45-degree line and lines representing relative densities of 70 and 100 percent

based on maximum and minimum densities given in Table 3.3 for twice cycloned Brenda sand are also shown in this figure. The results at or below 70 percent relative density show an extremely good correlation whereas those at higher densities show some scatter. However, in view of the large variations in densities that can occur at this site over short horizontal and vertical distances, particularly in areas of high densities, the scatter in the results is considered to be within an acceptable range of values. These variations in densities occur due to non-uniformity of compactive effort applied by the bulldozers used to form cells for hydraulic filling. Furthermore, the above test sites had been located in an area of the dam where a lift of sand fill had been placed by conventional earth moving equipment during an unusually long shutdown period due to a labor strike, hence the extremely high densities as shown in Figure 4.18. Figures 4.19 and 4.20 present photographs of the tripod and drop hammer arrangement, and a density sample recovered by the drive cylinder method respectively.

Figure 4.21 presents results from two typical test sites at the Brenda tailings dam where perched water table was encountered. One test site was located at the crest of the dam and the other at the toe. The results are shown in terms of dry density and degree of saturation computed from readings of nuclear probes. It is encouraging to note that within the zones of perched water table the degree

of saturation computed as above, in general, ranges from 98.0 to 102.5 percent, encouraging results indeed.

4.4.4.6 Results of Field Investigations

On the basis of the above considerations and the results of the previously described comparative testing it is concluded that the driving of access tubes produces satisfactory results. All further testing was carried out in access tubes installed by this method. Figures 4.22 to 4.25 present results of all further testing carried out at the three tailings dams included in this study.

Figure 4.22 presents typical density results from tests performed at Brenda dam at three locations - two at the crest of the dam and one at the toe. Approximate locations of the test sites are shown in the figure. In conjunction with sampling required for determination of grain size distributions, Standard Penetration Tests were performed at two of the locations and the results are shown. Results are also shown for a Dutch cone test performed at one of these locations during a previous investigation by Ripley, Klohn, and Leonoff (1973). Straight lines representing 30, 50 and 70 percent relative densities based on the limiting densities given in Table 3.3 for the twice cycloned Brenda sand are also shown. N-values measured at the two test locations (Figure 4.22) are shown plotted in Figure 4.23. For comparison, N-values computed

according to the correlation of Gibbs and Holtz (Figure 4.13) and Bazaraa (Equations 4.1) for relative densities measured at these locations by nuclear probe are also shown in Figure 4.23. It is obvious that the measured values do not correlate well with those computed by either method.

Figure 4.24 presents results of tests performed at the Bethlehem tailings dam. The relative density lines of 30, 50 and 70 percent based on limited densities given in Table 3.3 for 1972 Bethlehem sand are also shown.

Figure 4.25 presents results of tests performed at the Craigmont tailings dam at two locations - one at the crest of the dam and the other on the beach at a distance of 300 feet from the first.

4.4.4.7 Discussion of Results

Densities at the test locations shown in Figures 4.21 and 4.22 at the Brenda tailings dam as obtained by testing with the nuclear probes range from 84.0 to 106.0 pcf or 25 percent to over 100 percent, in terms of relative density. The majority of the test results, however, fall within a range of 30 to 70 percent relative density with large portion of results giving a relative density of more than 50 percent. It is felt that the above large variations in densities at Brenda tailings dam are the result of the hydraulic cell method of construction followed at the site.

It is interesting to note that with the nuclear methods, moisture as well as density profiles are obtained as shown plotted in terms of degree of saturation along with the density results. Moisture profiles are useful in locating perched water tables and zones of saturation as shown in Figure 4.21 at Brenda tailings dam.

Density results at the test locations shown in Figure 4.24 at the Bethlehem tailings dam fall within a range of about 45 to 68 percent in terms of relative density and 89 to 94 percent of maximum density as determined by the Standard Proctor Test (Table 3.3). No water tables or zones of saturation were encountered at this site. It is interesting to note, however, that at the test site in Figure 4.24A where sand was placed more than a year before testing was carried out, the degree of saturation is a maximum of about 32 percent. While at the test site in Figure 4.24B, where sand had been placed about a month before the testing, the degree of saturation ranges to a maximum of 55 percent.

Density results at the test locations shown in Figure 4.25 at the Craigmont tailings dam range in value from 92.5 to 120.0 pcf or between 80 and 100 percent of maximum density determined by the Standard Proctor Test. Results from this site are not presented in terms of relative densities due to the high percentage of fines in

the material rendering it unsuitable for determination of maximum density by vibratory methods. As expected due to the upstream construction at this site, zones of high degrees of saturation are encountered as shown in Figure 4.25.

4.5 In Situ Permeability Tests

4.5.1 General

It has been indicated in the previous chapter that the subject of permeability of tailings sands is a very basic one to the safe and economical design of tailings dams. The results of a laboratory testing program to study the influences of void ratio and grain size distribution on the coefficient of permeability of various tailings materals have been presented in Chapter III. In addition to these variables which can be controlled in the laboratory, the coefficient of permeability is also a function of stratification and structure of materials in situ, which in the case of tailings dams depend on the method of sand placement, degree of segregation and possible resulting non-homogeneities. In view of these considerations, it is generally thought that coefficient of permeability can best be determined by in situ testing. In conjunction with density measurements discussed in the previous section, field investigations also included measurement of in situ permeabilities at the three tailings dams. All pertinent details of the testing program are presented below.

4.5.2 In Situ Permeability Measurements Below Water Table

Below the ground water table, in situ permeability determinations can be made by two methods:

i) pumping-out test in which water is pumped from a bore hole or a test pit at a constant rate and the drawdown of water level observed in observation wells placed on radial lines at various distances from the pumped well, and

ii) infiltration test in which water is fed into a bore hole or a piezometer and the rate of seepage observed under a constant or variable head. A variation of this test is to bail out the water from the bore hole and observe the rate of inflow under constant head condition or observe the rise in water level as a function of time.

The pumping-out test is rather expensive to perform but the theory and the computational techniques are well established. Computational techniques are also well established in the case of infiltration tests but knowledge of a parameter called "intake or shape factor" of the bore hole or piezometer tip is required. The intake factor depends on the shape and the dimensions of the seepage area of the bore hole or the piezometer tip. Intake factors for piezometers of various shapes have been presented by Hvorslev (1951) and are reproduced here in Figure 4.26.

Cases 1, 4 and 8 are of particular interest in this study and are discussed below.

The formula for Case 1 is based on the consideration of a point source. It is the same as that derived by Gibson (1963) for a spherical piezometer tip by direct integration of the governing equation. The formula for Case 4 does not have any formal mathematical basis and is empirical in nature. The derivation of the formula for Case 8 is based on flow from a line source for which the equipotential surfaces are semi-ellipsoids (Dachler, 1936). Therefore, the formula provides only approximate results when it is applied to a cylindrical piezometer. Intake factors for cylindrical piezometers have been obtained by Al-Dhahir and Morgenstern (1969) by numerical integration of the governing equation using the finite difference technique.

4.5.3 In Situ Permeability Measurements Above Water Table

4.5.3.1 General

Whereas theory and procedures for performing in situ permeability tests in saturated soils below the ground water level are well established as described in the previous section, the opposite is true in the case of partly saturated soils above the water table. Generally this presents little problem in geotechnical engineering, as measurement of permeability of partly saturated soils is

seldom needed. This is not so in the case of a tailings dam, however, where generally a large volume of material in the dam is in a partly saturated condition and the zones of saturated material are inaccessible for testing. Furthermore, assessment of permeability of a tailings sand is required not only in a fully saturated condition but also under other conditions when the material may not be fully saturated such as on rewetting. All pertinent details of the studies carried out on the subject are presented below.

4.5.3.2 Previous Studies

A review of the English literature on the subject indicates that several investigators have reported work relevant to the topic at hand.

Aronovici (1955) and Johnson (1963) report a ring infiltrometer test in which two or more concentric pipes are pressed or driven into the soil for the purpose of maintaining a small head of water on a given surface area. It results in the measurement of rate of infiltration under the influence of gravitational head and hence, the coefficent of permeability in the vertical direction. The test, however, can only be performed at shallow depths below the ground surface.

Schmid (1966) suggests a variable head test in a cased hole above the ground water table assuming that the

wetting front has a spherical shape. The results from this test are likely to be approximate, at best, as the flow from a cased hole above the ground water table is of an axisymmetric nature rather than spherical as will become obvious later in this development.

Zangar (1953) proposes a "pumping-in" test in which a constant discharge of water is fed into a 6-inch diameter or larger auger hole with a gravel pack at the bottom. The water level in the casing will rise until the head is sufficient to transport flow introduced. The interpretation of the results is based on the work of Glover (1953). Glover summarizes the problem as follows,

> "Since the gravitational potential must be treated explicitly here, an exact solution would require that an expression be found which would satisfy Laplace's equation within a region possessing radial symmetry with respect to the axis of the hole. At an inner boundary the pressures would have to be adjusted to zero along some streamline with the gravitational potential accounted for everywhere.
>
> It would be very difficult to find a solution satisfying these requirements, and it will therefore be expedient to use an approximation. This will be obtained through the following procedure. Consider first the case where the surface of the ground is kept supplied with water so that it remains covered to a very small depth. The water will then move downward through the ground under the influence of gravity and the pressure will be zero everywhere. The flow will be at a rate which could be maintained by a unit pressure gradient. Now suppose a point-source of strength q second-feet (ft^3/sec) is superimposed on the gravitational flow system. This will give rise to pressures and new velocities. At a great distance below the source, the velocities due to the source will be negligibly small and only the velocities due to the gravitational force will remain ....."

He proposes a solution using a series of point sources of increasing strength with depth along the axis of the hole. Glover's solution satisfies the Laplace Equation but some of the boundary conditions are not satisfied.

A "pumping-in" test can also be performed by discharging water into a cased hole under a constant pressure. The rate of discharge will vary with time, ultimately reaching a constant value provided the test is continued for sufficient length of time. Palubarinova-Kochina (1962) solves the above axi-symmetrical flow problem by considering a point source in otherwise dry soil. The solution is approximate but satisfies the Laplace equation as well as all the boundary conditions. Since the solution is attributed to an unobtainable Russian paper and the brief presentation by Palubarinova-Kochina contains several errors, a detailed derivation has been prepared (Nuttall, 1973) and is included in Appendix C.

The velocity potential and the stream function for the flow problem at hand in cylindrical coordinates are:

$$\Phi = -\frac{Kq}{\sqrt{z^2 + r^2}} + \frac{Kq_1}{\sqrt{(b+z)^2 + r^2}} + Kz \qquad ..(4.4)$$

$$\Psi = \frac{Kr^2}{2} - \frac{Kqz}{\sqrt{z^2 + r^2}} + \frac{Kq_1(z+b)}{\sqrt{(z+b)^2 + r^2}} \qquad ..(4.5)$$

where $r^2 = x^2 + y^2$,

z is vertical axis (positive in downward direction as shown in Figure 4.27);

K = coefficient of permeability;

Φ = velocity potential; and

Ψ = stream function.

The potential function Φ and the stream function Ψ are defined with respect to velocity components V_r and V_z (Figure 4.27) by

$$V_r = \frac{\partial \Phi}{\partial r} = \frac{1}{r} \frac{\partial \psi}{\partial z} \quad \text{.....(4.6)}$$

and

$$V_z = \frac{\partial \Phi}{\partial z} = - \frac{1}{r} \frac{\partial \psi}{\partial r} \quad \text{.....(4.7)}$$

Φ is related to the piezometric head h and the pore pressure p by

$$\Phi = -kh = -k \left(\frac{p}{\gamma_w} - Z\right) \quad \text{.....(4.8)}$$

where γ_w = unit weight of water

The velocity potential given in Equation 4.4 represents a point source at the origin, and of strength

$Q = 4K¶q$,

a sink on the z-axis at $z = -b$ and of strength

$Q = -4\pi K q_1$, and a uniform field with velocity parallel to the positive z-axis.

On the basis of the derivation presented in Appendix C, Equations 4.4 and 4.5 can be rewritten as:

$$\Phi = Ka\left[-\frac{.78}{\left|\sqrt{Z^2 + R^2}\right|} + \frac{2.76}{\left|\sqrt{(Z+4.5)^2 + R^2}\right|} + Z\right] \quad \ldots(4.9)$$

$$\Psi = Ka^2\left[\frac{R^2}{2} - \frac{.78}{\left|\sqrt{Z^2 + R^2}\right|} + \frac{2.76\ (Z+4.5)}{\left|\sqrt{(Z + 4.5)^2 + R^2}\right|}\right] \quad \ldots(4.10)$$

where $a = .32\left|\sqrt{\frac{Q}{K}}\right|$,

$Z = \frac{z}{a}$, and

$R = \frac{r}{a}$.

Other details of the derivation presented in Appendix C are shown plotted in Figure 4.28. Lines of equal head are also shown.

4.5.3.3 Formulation Relevant to Constant Head Permeability Test Above Ground Water Table

From a constant head test in a saturated soil below the ground water level the coefficient of permeability K can be determined by:

$$K = \frac{q}{Fh} \quad \ldots\ldots(4.11)$$

where q = rate of discharge

F = intake factor (Figure 4.26)

h = excess head at the piezometer tip

It is hypothesized in this presentation that if a constant head test is performed above the ground water level until a steady state condition is reached as discussed in the previous section (Zangar, 1953, and Palubarinova-Kochina, 1962), the coefficient of permeability can be computed also from Equation 4.11. In order to compute the coefficient of permeability, a knowledge of intake factor F for the piezometer is therefore needed under the existing test conditions.

Initially, the soil around the piezometer is only partly saturated. As the test progresses, a zone of increased saturation develops around the piezometer tip until an ultimate shape as shown in Figure 4.28 is reached with an ever advancing wetting front below the piezometer. Ultimately within this wetted zone, a steady state seepage condition is reached and the distribution of piezometric head can be represented by the Laplace equation in cylindrical coordinates for an axisymmetrical case:

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\frac{\partial h}{\partial r} + \frac{\partial^2 h}{\partial z^2} = 0 \qquad .....(4.12)$$

Where h, r and z are as described in the previous section except z is positive in upward direction. The intake factor for the piezometer can be determined from a solution of Equation 4.12 for the appropriate boundary conditions as discussed below.

4.5.3.4 Numerical Analysis by Finite Element Technique

In this study, a solution of Equation 4.12 is obtained by a finite element technique for piezometers of spherical and cylindrical shapes, and for a cased drill hole. The finite element program used herein is that which has been presented by Kealy and Busch (1971) based on the work of Taylor and Brown (1967). This computer program can handle problems of two dimensional plane flow as well as those of axisymmetrical flow. The program is rather versatile and can analyze an anisotropic flow system as well as establish the location of the phreatic surface. The inner workings of this finite element program have been discussed in great detail by several authors (Kealy, and Busch, 1971; and Guther, 1972). Therefore only a brief description of the input information required and the output information produced by the computer program is given herein.

The first step in any numerical analysis is to define or isolate the region of interest in terms of boundary conditions. Figures 4.29A and 4.30A illustrate

typical input data required for analysis relevant to spherical and cylindrical piezometers. Boundaries are specified by the geometry of the problem as follows:

i) surface DEF represents applied upstream equipotential line along the surface of the piezometer;

ii) surface ABC represents an estimated initial free surface;

iii) surfaces AD and FG represent impervious boundaries due to axial symmetry of flow; and

iv) surface CG represents advancing wetting front and downstream zero potential line.

The fourth boundary condition along surface CG (Figure 4.29A and 4.30A) deserves a further comment. As discussed previously in Section 4.5.3.2, at a great distance below the piezometer, the pore pressures and the velocities due to the applied pressure at the piezometer tip are negligibly small and the water flows in a downward direction under the gravitational forces only. Therefore, if it can be assumed that surface CG is situated at a sufficient depth below the piezometer tip, then water flows across this boundary in a downward direction under a hydraulic gradient of unity with pore pressures equal to zero.

The above program develops and prints the follow-

ing information:

i) reprint of input data,

ii) nodal point pressures and equipotential values,

iii) element flow rates at the centre of each element, and

iv) for free surface problems, mesh correction is printed after each iteration. Figures 4.29B and 4.30B illustrate typical out-put data for the spherical and cylindrical piezometers. Points A' and B' on the Phreatic surface obtained by Palubarinova-Kochina's method for an equivalent point source are also shown. As can be seen, there is a remarkable agreement between the results from the two methods.

The rate of discharge q can be computed from the information in item (iii) above. Therefore from known values of h, K and q, the intake factor F can now be computed from Equation 4.11. For homogeneous and isotropic soil conditions, the results are presented in a convenient dimensionless form F/D in Table 4.3. For comparison purposes, F/D values are also presented for below-ground-water condition based on the works of Hvorslev (1951), Gibson (1963) and Al-Dhahir and Morgenstern (1969).

It is of interest to note that F/D values determined by the finite element analyses for above-ground-water condition are consistently higher than those based on Hvorslev's formulae (Figure 4.26) for comparable piezometers for a below-ground water condition. For a cylindrical piezometer with L/D ratio between 3 to 6, F/D values for above-ground-water condition are approximately one-third higher than those for comparable piezometers in below ground-water condition. It can be argued that this difference in F/D values is rather insignificant in light of the inaccuracies inherent in in situ permeability tests. The difference for a cased drill hole, however, appears to be significantly higher and hence should perhaps be taken into consideration.

For anisotropic soil conditions Hvorslev (1951) presents the following equations for flow through the intakes (Cases 4 and 8 in Figure 4.26) for the below-ground-water condition:

Case 4 $$q = 2.75\, D\, K_m h \qquad \text{.....(4.13)}$$

Case 8 $$q = \frac{2\pi L\, K_h h}{\operatorname{Ln}\left(m\, L/D + \sqrt{1 + (mL/D)^2}\right)} \qquad \text{.....(4.14)}$$

where $K_m = \sqrt{K_h K_v}$

$m = \sqrt{K_h / K_v}$

K_h = coefficient of permeability in horizontal direction.

K_v = coefficient of permeability in vertical direction.

A limited number of analyses were carried out to study the influence of anisotropy on intake factors for piezometers in the above-ground-water condition. The results are presented in Table 4.4.

4.5.3.5 Studies with Model Piezometer

A study was undertaken in the soil mechanics laboratories at the University of Alberta with a model piezometer as a check on the theory and the results of the analyses presented in the previous sections. A cylindrical piezometer of ½-inch diameter and L/D ratio of three with design details similar to those of the piezometer to be employed in the field investigations was used in the study. Design details of the piezometer used in field investigations are given in Appendix B.

Tests were carried out in the large drum-like container used previously in calibration of nuclear probes (Section 4.4.4.4). A tailings sand was placed under water and around the model piezometer situated in the centre of the drum at a depth of about 12 inches from the top of the container.To assimilate field conditions, the sand was then allowed to drain from the bottom with vacuum being applied as required to facilitate drainage. After sand had been drained sufficiently, the vacuum was disconnected and the average void ratio of the sand determined. The piezometer standpipe was filled with water and connected to a constant

head permeability apparatus to be described later in this chapter in conjunction with field testing. In this manner, the test was allowed to proceed under a constant head condition and the rate of discharge recorded with time. The test was allowed to continue until a steady state condition was reached or at least nearly so. To ascertain the degree of reproducibility possible, the test was repeated several times under different heads. The results are discussed below.

As discussed previously in Section (4.5.3.2) in the above constant head test, the rate of discharge has a high initial value and decreases with time, ultimately reaching a constant value after a long time. This ultimate constant rate of discharge can then be used in Equation (4.11) to compute the coefficient of permeability. To get a constant rate of discharge, however, a test may have to be continued for several hours, which is not always convenient, particularly in the field. This will become obvious later in this chapter. Typical discharge versus time plots are shown in Figure 4.31. It can be seen in this figure that the rate of discharge drops off rather rapidly in early stages of the test from the initial high value and then decreases more gradually at a reduced rate thereafter. The results are also shown plotted in Figure 4.32 in terms of q versus $t^{-\frac{1}{2}}$. These appear to result in linear plots and are in general agreement with the work of Al Dhahir

(1967) and the following rather well established relationship from the theory of infiltration in unsaturated soils by Philip (1957):

$$Q = At^{1/2} + Bt \qquad .....(4.15)$$

where Q is cumulative infiltration, A and B are constants, and t is time since infiltration commenced; and, the rate of infiltration,

$$q = \frac{dQ}{dt} = \tfrac{1}{2}At^{-1/2} + B$$

or $$q = A't^{-1/2} + B \qquad .....(4.16)$$

The linear nature of the q versus $t^{-1/2}$ plots is used to an advantage in estimating ultimate q_∞ at t_∞ by extrapolating the results to $t^{-1/2} = 0$ axis. The q_∞ is then used in Equation (4.11) to compute the coefficient of permeability.

In situ permeability values measured during the model piezometer studies are presented in Table 4.5. For comparison purposes, the values of permeability determined in the laboratory permeameter as discussed in Chapter III for the tailings sand used in the above studies are also given in Table 4.5. At void ratios obtained in the model tests, the permeability values from model piezometer tests range from 3.7×10^{-3} to 5.5×10^{-3} cm/sec whereas those from permeameter tests range from

11×10^{-3} to 12×10^{-3} cm/sec. These results indicate that permeability values from the model piezometer tests are one-third to one-half of those obtained by permeameter tests. This apparent discrepancy in the results is either due to the differences in degrees of saturation or the differences in the soil structure being obtained in these two types of tests or both. In view of these possible variations, the above discrepancy is not of any concern.

It is felt that the results from the model piezometer tests represent the effective coefficient of permeability under the prevailing test conditions, similar to a large extent to those likely to exist in the field. The measured values may, therefore, be considered somewhat indicative of those which are to be measured in the field, all other things being equal.

In the last test, food dye was added to the water supply in an attempt to establish the shape of the wetted zone around the piezometer. Figure 4.33 is a photograph of a cutaway section illustrating the same. It should be pointed out that this figure is presented to show the general shape of the wetted zone which certainly confirms the results of the theory and the finite element analyses but no quantitative conclusions can be drawn as the introduction of dye tended to plug up the sand with time. It also confirms that the test container was

sufficiently large so that the boundaries of the wetted zone were not interfered with.

4.5.3.6 Field Equipment and Test Procedure

A constant head permeability apparatus was designed for use in the field. A schematic sketch illustrating the details of this apparatus is presented in Appendix B. A noteworthy feature of the apparatus is that it is equipped with a series of flow meters so that the rate of discharge can be recorded at any instant of time by observing the position of the indicator in an appropriate flow meter. A photograph of the above equipment while in field use is shown in Figure 4.34. A 5-micron filter was provided in the system to prevent the fine suspended matter in the water from entering the piezometer and possible plugging of soil. Figure 4.35 is a photograph of a badly plugged filter after being used in a permeability test and another one in new condition.

A 1.5-inch diameter brass piezometer with L/D ratio of three has been designed for use in the field. The details of the piezometer are illustrated in Appendix B. The piezometer is essentially similar to that reported by Parry (1971) in that it has a steel sleeve with a conical point covering the porous element during driving. The outside diameter of the sleeve is 1.62 inches and the piezometer is attached to a 10-foot length of A-rod with outside diameter of 1.72 inches. The piezometer is driven

into the ground using the drop hammer and tripod arrangement described in Section 4.4.4.5 for driving density tubes (Figure 4.19). After the first 10-foot length of A-rod is driven, 5 foot extensions of E-rod are added as required to achieve the desired penetration. All rod couplings have been suitably modified to provide pressure tight O-ring connections and to give a minimum inner diameter of 3/4 inches so that head loss in the pipe during a test can be kept to a minimum.

After the piezometer tip is driven to the desired depth, the rods are retracted through a distance of approximately 6 inches to withdraw the porous element from its protective sleeve. Figure 4.35 is a photograph of the piezometer rods immediately after being retracted.

The piezometer standpipe is then filled with water and connected to the constant head permeability apparatus and the test is continued in a manner similar to that used in the model piezometer studies. Except for some of the tests at very shallow depths or where high rates of discharge (in excess of 1 usgpm) were encountered while filling the piezometer, all tests were performed in the manner described above. At these shallow depths and/or areas of high rates of discharge, a constant rate of discharge was fed into the piezometer standpipe and the rise in water level was recorded with time until a constant

(or at least nearly so) water level was established. Head versus $t^{-\frac{1}{2}}$ plot (Figure 4. 37) resulted in a linear relationship and h_∞ was obtained by extrapolation as in the case of q versus $t^{-\frac{1}{2}}$ plots (Section 4.5.3.5).

4.5.3.7 Field Testing and Presentation of Results

In conjunction with work described in Section 4.4.4.5, a preliminary testing program was undertaken in the fall of 1972 to assess the suitability of the permeability equipment under actual field conditions. Tests were performed at all three dam sites. The test results from Brenda, however are considered unreliable due to the inexperience with the equipment at the time of the testing and hence, are not included herein. Test results from Bethlehem and Craigmont dams are summarized in Table 4.6. As can be seen, results of duplicate tests show a remarkably high degree of reproducibility.

The above testing was restricted to shallow depths only. Subsequent testing was undertaken in 1973 to extend the in situ permeability measurements to greater depths (a maximum of 62 feet below ground surface). Tests were performed at selected depths at 2 to 3 typical locations at each dam site. The results are presented in Figures 4.21A, 4.22, 4.24 and 4.25.

4.5.3.8 Discussion of Results

The results from Brenda tailings dam are shown in Figures 4.21A and 4.22 and range from 1.9×10^{-2} to 2.5×10^{-5} cm/sec. Figure 4.24 presents results from Bethlehem dam ranging from 7.0×10^{-3} to 0.4×10^{-3} cm/sec; and Figure 4.25 shows results from Craigmont dam ranging from 1.8×10^{-4} to 0.4×10^{-4} cm/sec.

Generally speaking, there are four variables which can influence the permeability values as measured at these dams - grain size distribution, density, structural details such as stratification, anisotropy, non-homogeneities, etc., and degree of saturation obtained during a test. Density variations at the test locations are shown in the above figures. Typical samples were collected from various depths during the field investigation and grain size determinations made in the laboratory. Results are shown plotted in Figure 4.38. An approximate idea of the range of variations in grain size distributions can be obtained from these results. The structural details are essentially a function of the method of sand placement followed at a dam site and cannot be assessed quantitatively. Similarly, degree of saturation obtained during a test cannot be evaluated.

At Bethlehem dam measured permeability values range from (Figure 4.24) 7.0×10^{-3} to 0.4×10^{-3} cm/sec. A

review of this figure indicates that densities at this site range from 85.5 to 91.5 pcf. For the tailings sand tested in the laboratory, permeability values for this range of densities vary between 3.0×10^{-3} and 0.9×10^{-3} cm/sec. In view of these results and the possible variations in grain size distribution of materials (Figure 4.38), the measured values of 7.0×10^{-3} to 0.4×10^{-3} cm/sec for the coefficient of in situ permeability are indeed encouraging.

In view of the expected large variations in grain size distribution of materials at Craigmont dam due to the upstream construction and as shown in Figure 4.38, the measured permeability values of 1.8×10^{-4} to 0.4×10^{-4} cm/sec are within a remarkably narrow range.

Measured permeability values at Brenda dam show the largest range (1.9×10^{-2} to 2.5×10^{-5} cm/sec). The sand at this site is twice cycloned and by far the coarsest and the cleanest of the materials encountered at the three dams investigated during this study. Whereas the values at the upper end of the range appear to be entirely reasonable for the sand in question, the values at the low end of the range appear rather low. Figure 4.38 shows the grain size distribution curves for samples obtained from site c in Figure 4.22. On the basis of the laboratory test results shown in Figure 3.10 and the test results

from Bethlehem dam, a range of permeability values of 2.0×10^{-2} to 3.0×10^{-4} cm/sec can be justified for the above range of grain size distribution and the densities shown in Figure 4.22.

A close examination of the results in Figures 4.21A and 4.22 indicates that conditions encountered at these locations can be subdivided into three categories:

i) areas of nominal densities (under 50% relative density) at shallow depths, typified by depths to 25 feet in Figure 4.22A and to 13 feet in Figure 4.22C;

ii) areas of high densities (in excess of 60% relative density) and full or nearly full saturation typified by depths between 8 to 17 feet in Figure 4.21A; and

iii) areas of high densities and low saturation, typified by depths below 25 feet in Figure 4.22A and below 13 feet in Figure 4.22C.

Within areas in category (i) above, the measured permeability values range from 1.9×10^{-2} to 2.9×10^{-3} cm/sec - a reasonable range of values for the sand in question.

Within areas in category (ii), the measured permeability values are as low as 1.6×10^{-4} cm/sec - some-

what low but perhaps still acceptable in view of the values obtained at other sites. Within areas of the third category, the measured values of permeability range as low as 2.5 x 10^{-5} cm/sec - extremely low value for the sand in question. It is surmised that the soil structure in these areas must be such that deairing is inhibited during a test resulting in low degrees of saturation around the piezometer tip and hence, the low permeability values. However, it should be recognized that only a limited number of tests were performed as part of this study and the results presented herein are not by any means conclusive. It can, however, be said that at Bethlehem dam where density conditions are fairly uniform due to the on-dam cycloning, the meausred permeability values fall within a reasonable narrow range and that wide variations in measured permeabilities at Brenda dam reflect the nonuniformity in the sand fill due to the irregular compaction being applied as part of the hydraulic cell method.

4.6 In Situ Pore Pressure Measurements in the Slimes

4.6.1 General

For C_V values (coefficient of consolidation) of slimes given in Figure 3.31 and the high rates at which slimes are likely to be deposited at some of the larger mines (for example Bethlehem and Brenda), analyses according to Gibson (1958) indicate that the slimes are likely to be underconsolidated with high excess pore pressures. This

will be dealt with in more detail in the next chapter. A field testing program was undertaken to measure in situ pore pressures in the slimes. Pore pressure measurements were taken at Bethlehem and Brenda ponds. No such measurements were made at Craigmont mines as the pond was inaccessible for testing with the equipment available.

In addition to the above measurements, remolded samples were obtained with a simple piston sampler to limited depths. Estimates of in situ density were made at selected test locations from recovered samples and also from measurements taken with the nuclear (depth-moisture) probe. All pertinent details follow.

4.6.2 Equipment and Test Procedure

All testing was carried out from a floating raft. A Geonor-type piezometer was used for the purpose. It consists of a 30 mm diameter cylinder of porous bronze connected to a central shaft by top and bottom end pieces. The bottom piece is conical in shape and the top is threaded to fit standard E-size drilling rod. The porous element is connected to a ¼ inch plastic tubing.

Initially, one end of the plastic tubing was connected to the piezometer and the other end pulled through a sufficient number of 5-foot sections of E-rod. The piezometer was held under water and the system deaired,

filled with water and the top end of the plastic tube attached to a pressure gauge mounted on a board placed on the raft. Extensions of E-rods were added as required and the piezometer pushed into the slimes to the desired depth. Pore pressure readings were recorded from the pressure gauge with time until a constant value was achieved. This usually required a period of several hours and quite frequently, the test was allowed to stabilize overnight. Pore pressure measurements were made by this method to a maximum depth of 80 feet below the surface of the slimes. Figure 4.39 is a photograph of the raft and the set up for measurement of pore pressures used in this testing.

4.6.3 Presentation of Results

At Bethlehem tailings pond, pore pressure measurements were made at selected depths at a site located approximately in the middle of the pond. Results obtained are presented in Figure 4.40.

Pore pressure measurements were made at Brenda tailings pond at three test locations approximately in the middle of the valley as shown in Figure 4.41. In situ density measurements were made at test location P-2 with the aid of the nuclear moisture probe. The results obtained are shown plotted in Figures 4.42 and 4.43. Samples were obtained from selected depths at all three test locations and water content determinations were made; the results were

essentially similar at equivalent depths for all test locations. In situ densities were computed from the above water contents; for ease of presentation, however, only results from test location P-2 are shown plotted in Figure 4.42.

4.6.4 Discussion of Results

Measured pore pressures are shown plotted as a function of depth in Figures 4.40A, 4.42A and 4.43. Straight lines representing hydrostatic pore pressures are also shown in these figures. Wet densities (γ_t) as computed from readings taken with nuclear moisture probe and from water content determinations made on the recovered samples are presented in Figures 4.40B and 4.42B. Although there is an apparent scatter in the density results yet for the purpose of demonstrating the order of magnitude of measured excess pore pressures, it appears reasonable to use average linear variations of wet density with depth as shown in these figures. Overburden pressures have been computed using the above linear variations of densities at the depths where pore pressures were measured. The results are plotted in Figures 4.40C and 4.42C. Hydrostatic pore pressures are also plotted in these figures against overburden pressures. It is interesting to note that unlike most cases in soil mechanics, hydrostatic pore pressures plotted against overburden pressures produce non-linear plots. This results from the fact that wet density

of slimes is not constant but varies with depth.

For comparison purposes, a 45-degree line is shown in Figures 4.40C and 4.42C. It can be seen in Figure 4.40C that the results from Bethlehem tailings pond plot essentially along the 45-degree line indicating that the slimes at this site have hardly undergone any consolidation to a depth of 80 feet where estimated total depth of slimes is 150 feet.

Similarly, results from test location P-2 at Brenda tailings pond indicate little or no consolidation to a depth of about 25 feet where estimated total depth of slimes is about 70 feet. The results indicate an increasing degree of consolidation with depth below the 25 foot level as shown in Figure 4.42C.

Results at test location P-3 (Figure 4.43A) at Brenda Mines indicate a similar pattern to that described above for P-2. On the other hand, at test location P-4 which is situated closest to the crest of the dam (at a distance of about 950 feet), it was not possible to push a piezometer below the depth of 17 feet under the weight of three men and the values measured indicate excess pore pressures of only slight magnitude (Figure 4.43B). A higher degree of consolidation is to be expected in the slimes near the dam at Brenda Mines for reasons discussed below.

It appears that the combined overflow from the cyclones (designated Brenda slimes in Figure 3.25) contains up to 30% sand sizes where samples recovered from the pond generally contain less than 5% sand (see for example results of sample from P-2 in Figure 3.25). This indicates that the large portion of sand and coarse silt sizes must drop out on the beach. The length of the beach can vary according to the time of year. Furthermore, from time to time total tailings are discharged into the pond, introducing more coarse material in the beach area. On the basis of the above observations, it can be concluded that coarser material or at least layers of it are present in the beach area, which will tend to accelerate drainage and hence, the larger degree of consolidation.

TABLE 4.1 Summary of Preliminary In Situ Density Tests Performed at Brenda Dam in 1972

Test Site	Depth - ft	Dry Density - pcf: Nuclear Method	Dry Density - pcf: Conventional Densometer
A-1*	4½	86.5	
	5½	85.5	
A-2*	4	89.5	
	5	85.0	
A-3*	4½	87.5	
	5½	88.5	
B	2	83.5	82.7
	6	83.0	
C	2	82.0	
	6	84.0	
	10	83.5	
	15	85.5	
D	2	83.5	83.3
	5	88.0	
	6	86.0	
	9	84.0	
	10	88.0	
	13	89.0	
	14	84.5	

* Sites A-1, A-2 and A-3 were located at distances of about 5 feet from each other.

TABLE 4.2 Summary of Laboratory Density Results Before and After Driving of Access Tube

Trial No.	Parameter	Initial by Measurements	After Driving Tube by Nuclear Probes
I	γ_t - pcf	110.0	111.5
	*w - %	29.7	29.7
	γ_d - pcf	84.7	86.0
II	γ_t - pcf	95.8	95.5
	w - %	11.0	11.0
	γ_d - pcf	86.3	85.5
III	γ_t - pcf	98.5	97.3
	w - %	13.1	13.1
	γ_d - pcf	87.2	86.0

* Water content was determined by measurements with the nuclear probes at the end of the test.

TABLE 4.3 Intake Factors for Piezometers
(Isotropic and Homogeneous Soil Conditions)

		F/D				
		Below Ground Water Level			Above G.W.L.	
Shape of Piezometer	L*/D	Hvorslev (1951) Figure (4.26) (1)	Gibson** (1963) (2)	Al-Dhahir and Morgenstern (1969) (3)	Finite-Element Analysis This Study (4)	% Difference Between (1) and (4)
spherical		6.3 Case 1	6.3	-	7.0	10%
cylindrical	3	10.4	10.9	11.0	13.5	30%
	6	15.3 Case 8	15.4	16.5	20.6	34%
cased drill hole (Case 4 - Figure 4.26)		2.75 Case 4	3.1		5.3	90%

* L/D is length to diameter ratio.
** F/D computed using sphere of equivalent surface area as suggested by Gibson (1963)

TABLE 4.4 Intake Factors for Piezometers (Anisotropic Soil Conditions)

Shape of Piezometer	L/D	m*	F/D: Below G.W.L. Hvorslev (1951) 1	F/D: Above G.W.L. F E Analysis This Study 2	% Difference Between 1 - 2
			With Respect to K_h		
cylindrical	3	1	10.4	13.5	30
		2	7.6	9.9	30
		3	6.5	-	
Hvorslev Case 8	6	1	15.4	20.6	34
		2	11.9	15.5	30
		3	10.5	-	
			With Respect to K_m**		
Hvorslev Case 4		1	2.75	5.3	90
		2	2.75	6.0	120
Cased drill hole		3	2.75	7.4	170

* $m = (K_h/K_v)^{1/2}$

** $K_m = (K_h K_v)^{1/2}$

TABLE 4.5 Summary of Results From Model Piezometer Studies

	Model Piezometer			Permeameter*
Void Ratio	Test No.	Head	K-cm/sec x 10^{-3}	K-cm/sec x 10^{-3}
.97	1	24"	3.7	12
.97	2	29"	5.5	12
.94	3	25"	4.0	11
.94	4	15"	5.0	11

* From Figure 3.10

TABLE 4.6 Summary of Permeability Results from Preliminary Field Tests

Dam Site	Test No.	Depth Below Ground Surface ft	K-cm/sec	Remarks
Bethlehem	A-1*	8	2.8×10^{-3}	Test locations
	A-2	8	3.1×10^{-3}	approximately
	A-3	8	3.9×10^{-3}	5 ft apart
	B-1*	8	3.0	Test locations
	B-2	8	3.2	5 ft apart
Craigmont	A-1**	9½	1.5×10^{-4}	A-1 and A-2
	A-2	9½	1.6×10^{-4}	approximately
	A-2	11½	3.4×10^{-4}	5 ft apart
	B-1**	10	3.4×10^{-4}	Water level at
	B-1	12½	4.7×10^{-4}	7½ ft below ground surface

* Test Sites A and B were located approximately 100 ft apart along and 30 ft upstream of the crest of the dam.

** Site A was situated at a distance of 30 ft upstream of crest of dam and Site B 150 ft upstream of Site A.

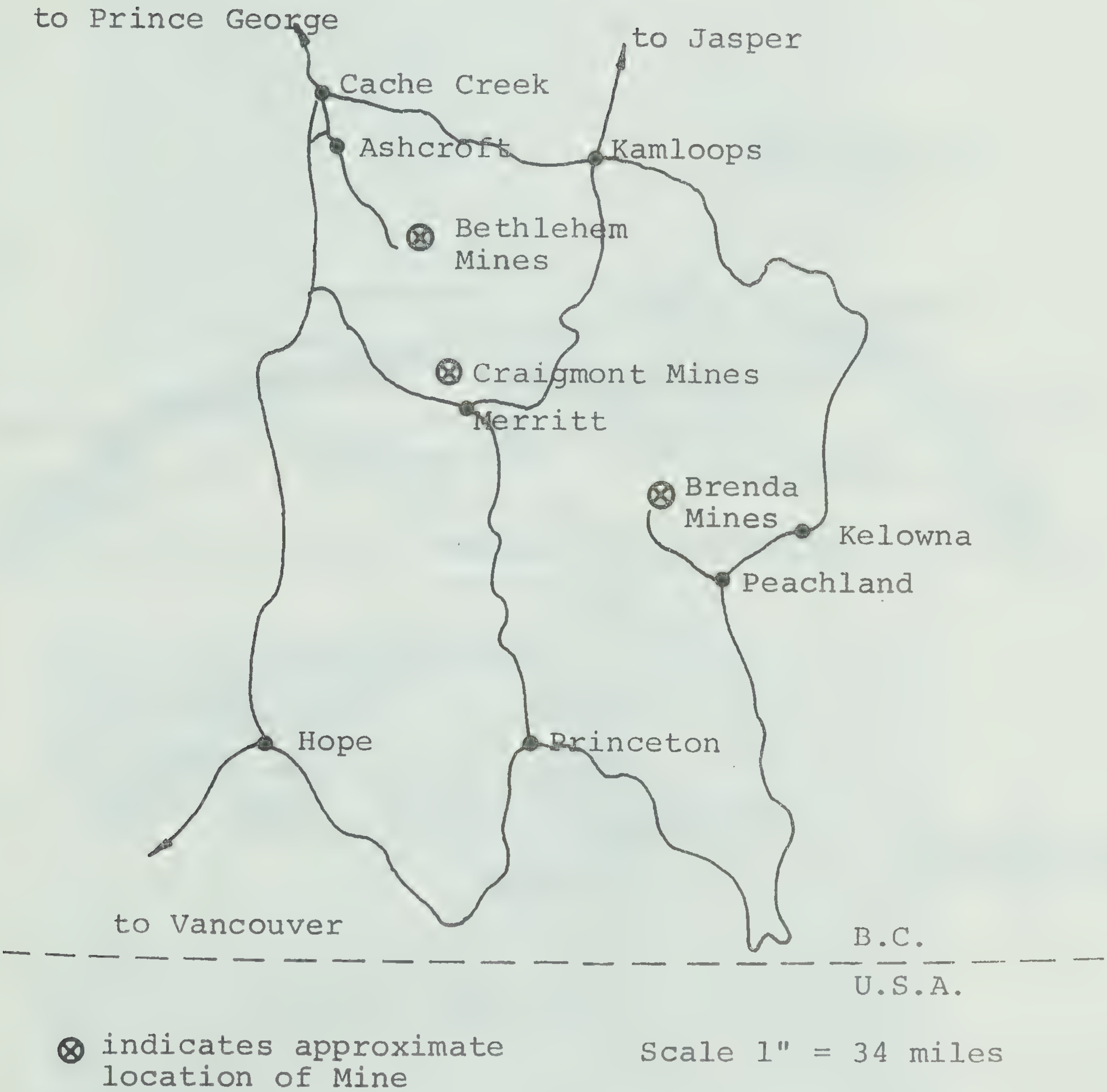


Figure 4.1 Map of British Columbia Showing Mine Locations

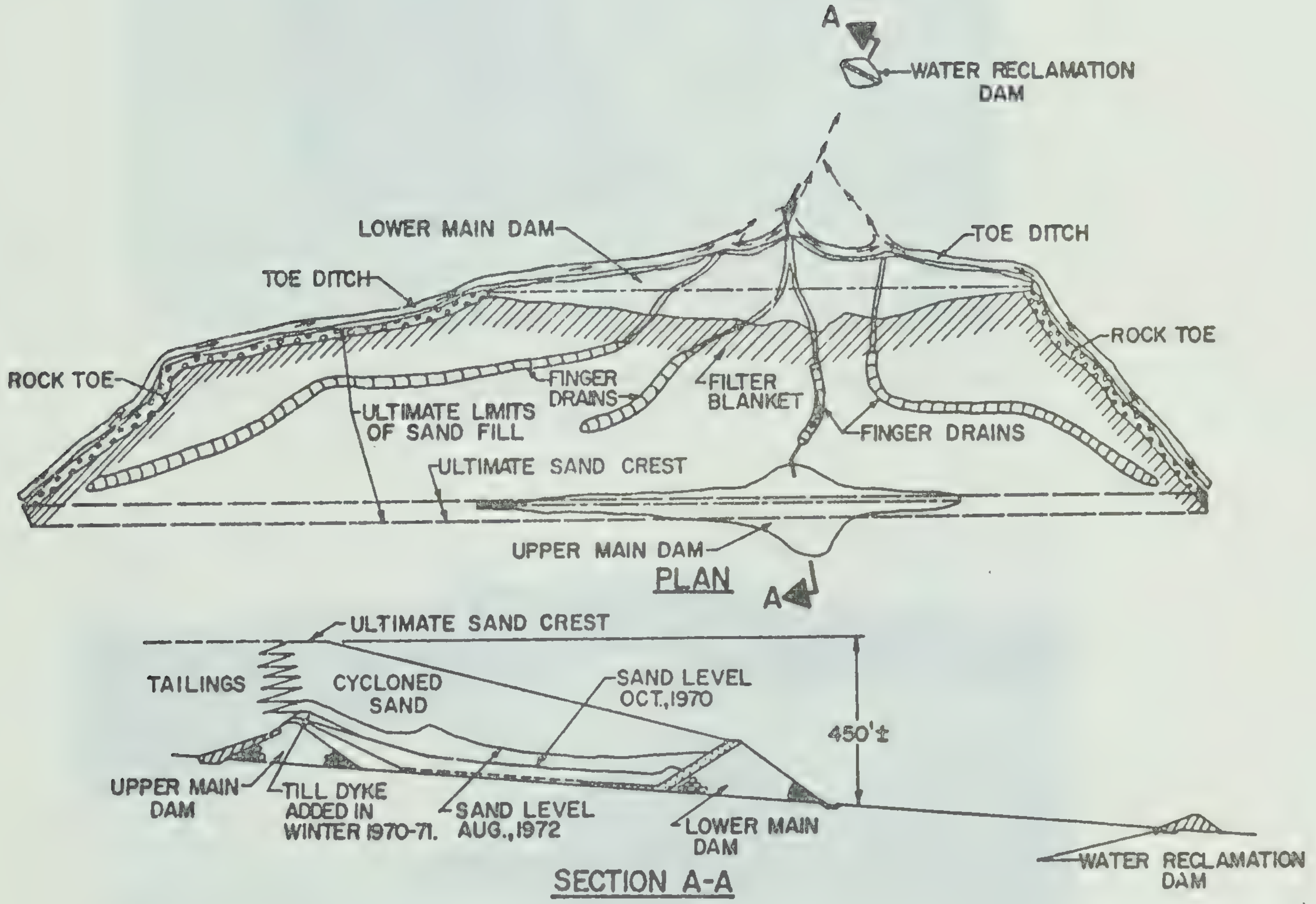


Figure 4.2 Brenda Tailings Dam (After Klohn and Maartman, 1973)

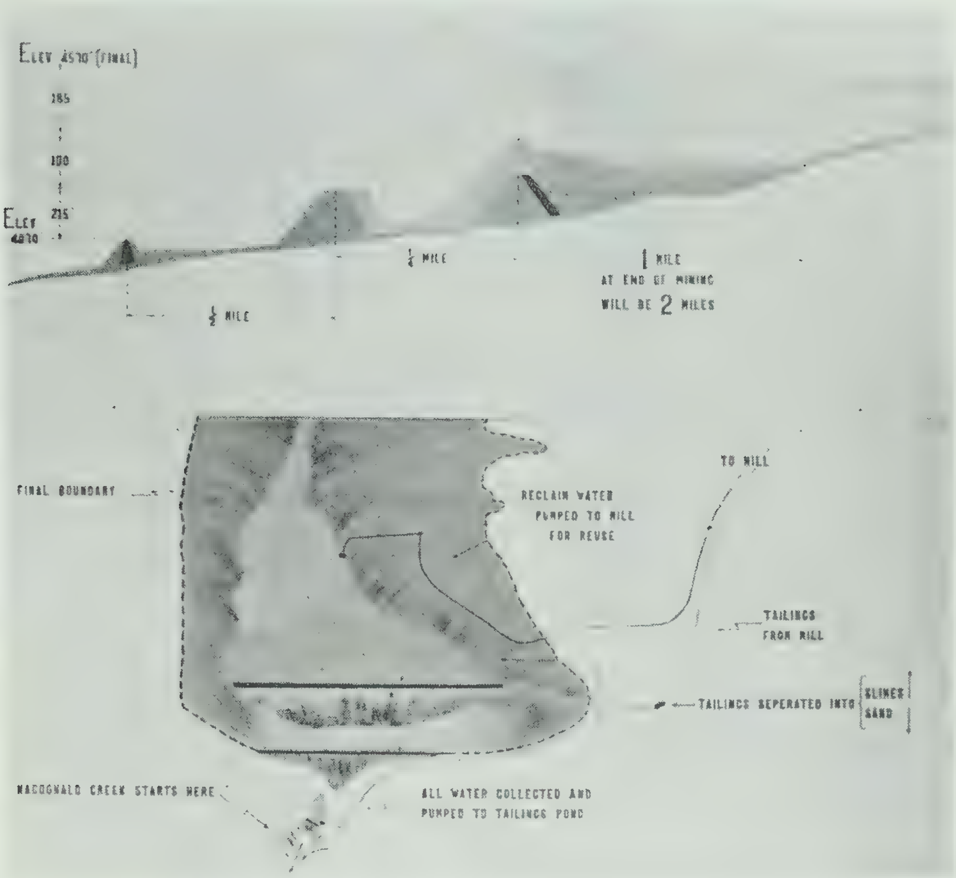


Figure 4.3 Tailings Disposal Concept at Brenda Mines

Figure 4.4 Brenda Tailings Dam June/1973

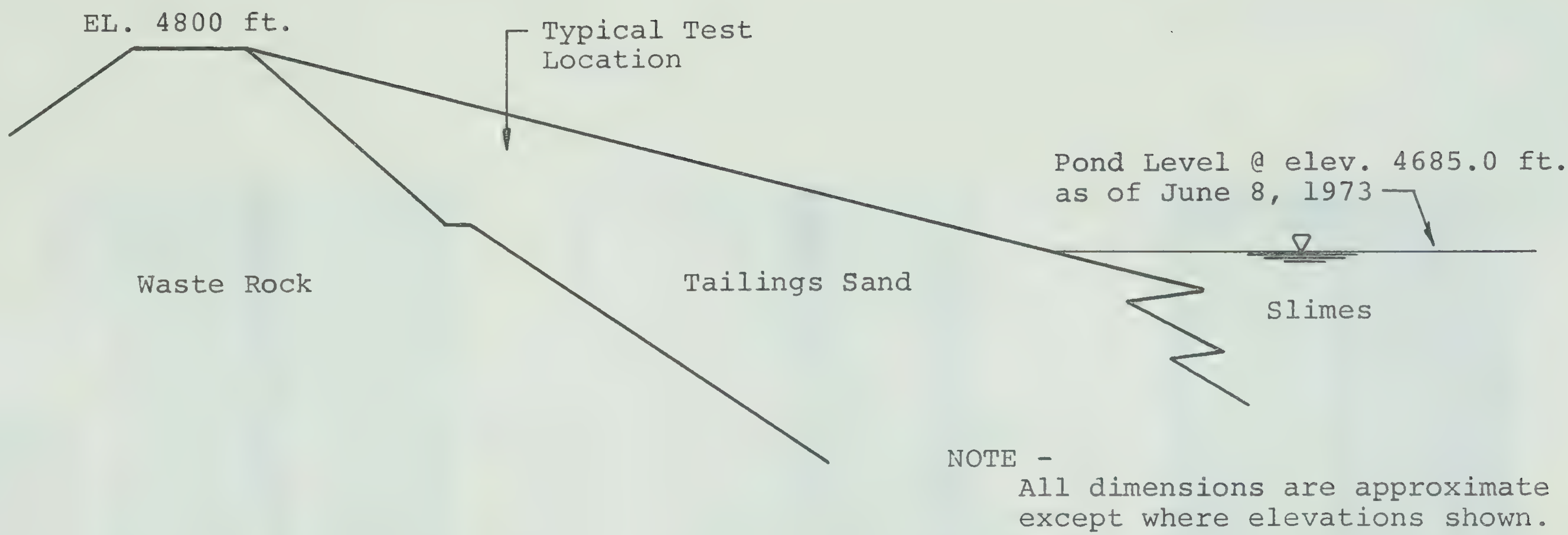


Figure 4.5 - Typical Section of Bethlehem Tailings Dam at Test Locations Situated at Approximately 1000 feet From Each End of the Dam.

Figure 4.6 Bethlehem Tailings Disposal Area
Photograph taken from near the barge pumps at the back of the pond.

Figure 4.7 Bethlehem Tailings Dam - South End
(Photograph taken from a boat on the pond.)

Figure 4.8 Bethlehem Tailings Dam - North End
(Photograph taken from a boat on the pond.)

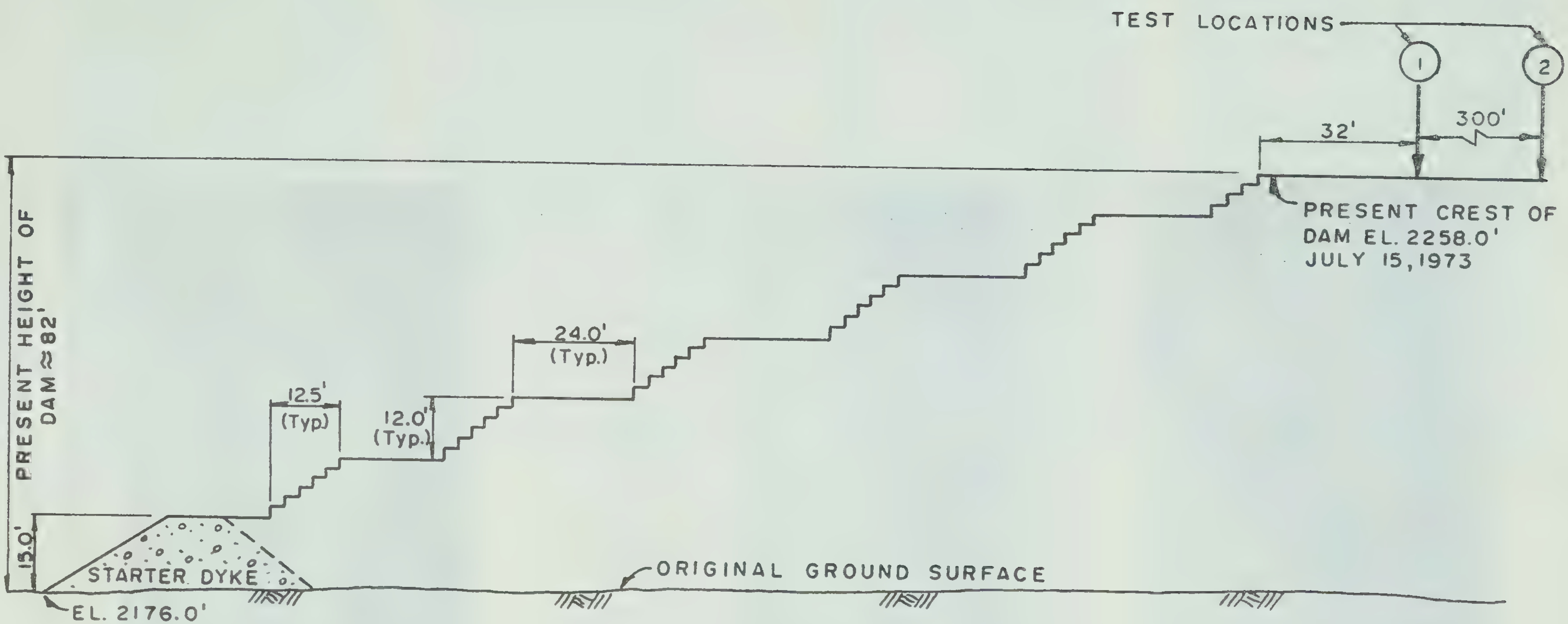


Figure 4.9 Typical Section Craigmont Tailings Dam at Test Locations Situated Approximately 1130 feet From East End of Dam

Figure 4.10 Brenda Tailings Dam - Hydraulic Cell Construction
A - Cell ready for sand placement
B - Cell during sand placement

Figure 4.11 Craigmont Tailings Dam

A - Spigotting from along the timber forms

B - Excess tailings discharged from overhead pipes.
Note - back ponding

C - Note cracking of slimes in back ponded areas during off season

D - Close up of cracked slimes.

Figure 4.12 Craigmont Tailings Dam - Method of Construction

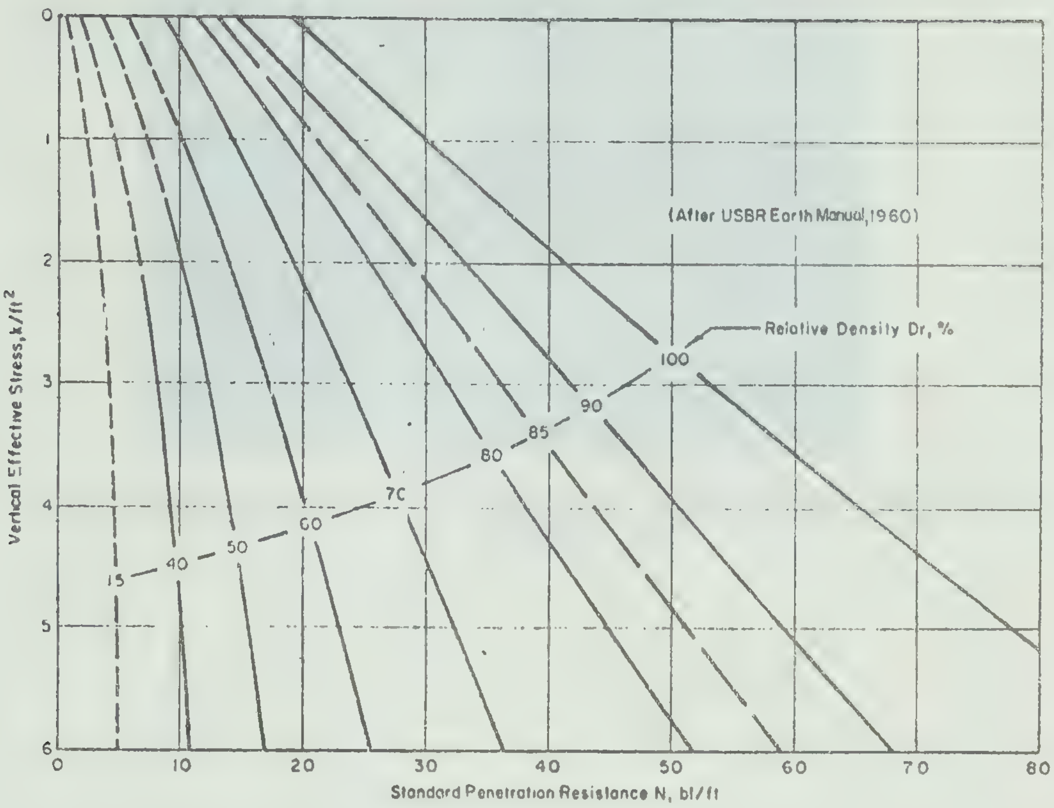


Figure 4.13 Correlation between relative density and standard penetration resistance (After Gibbs and Holtz, 1957)

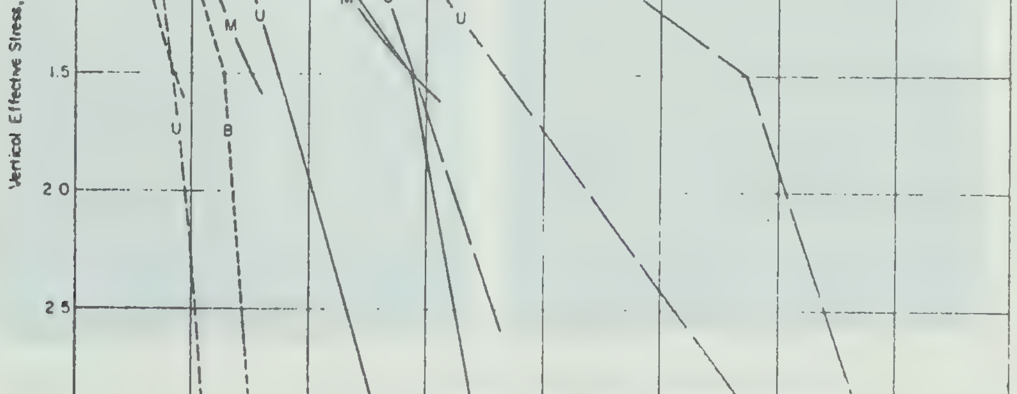


Figure 4.14 Comparisons of several correlations between relative density and standard penetration resistance (After Lacroix and Horn, 1973)

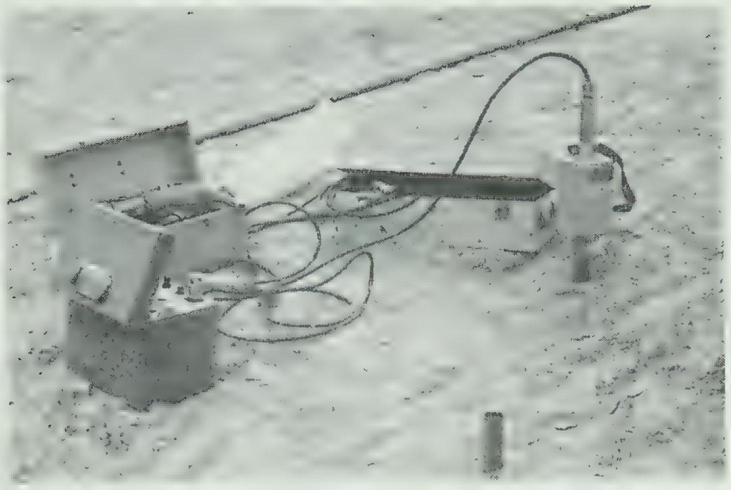


Figure 4.15 Nuclear Equipment during field use.

See next page for Figure 4.16.

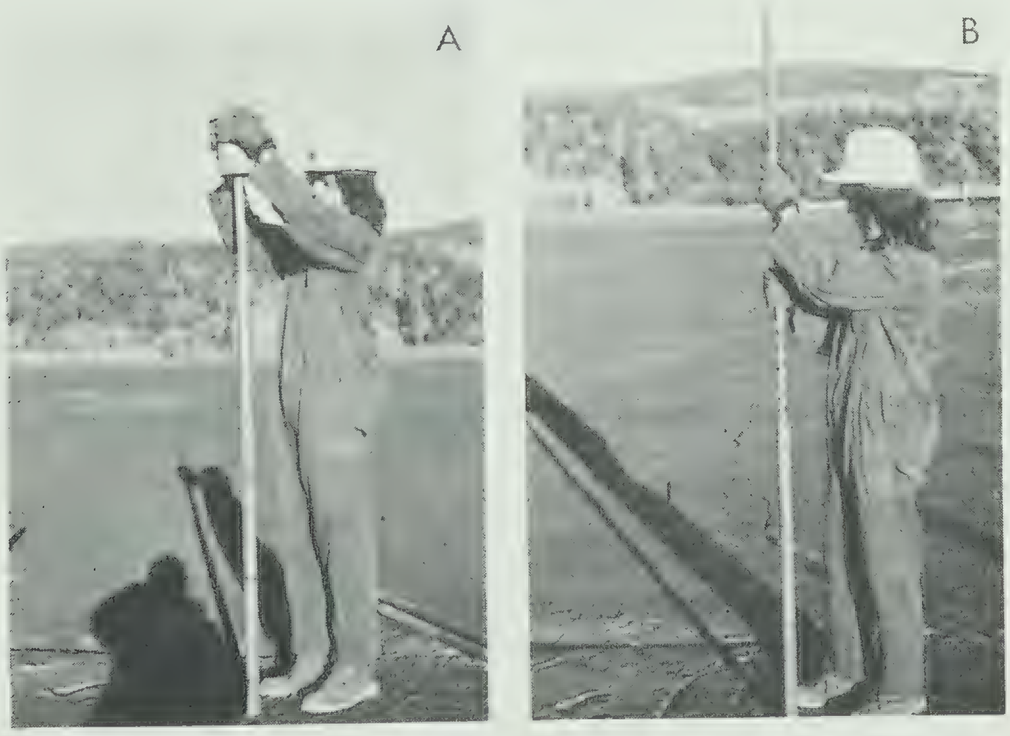


Figure 4.17 Tamp and Auger Method of Installing Access Tubes
A - Tamping
B - Lowering auger inside the tube between tampings.

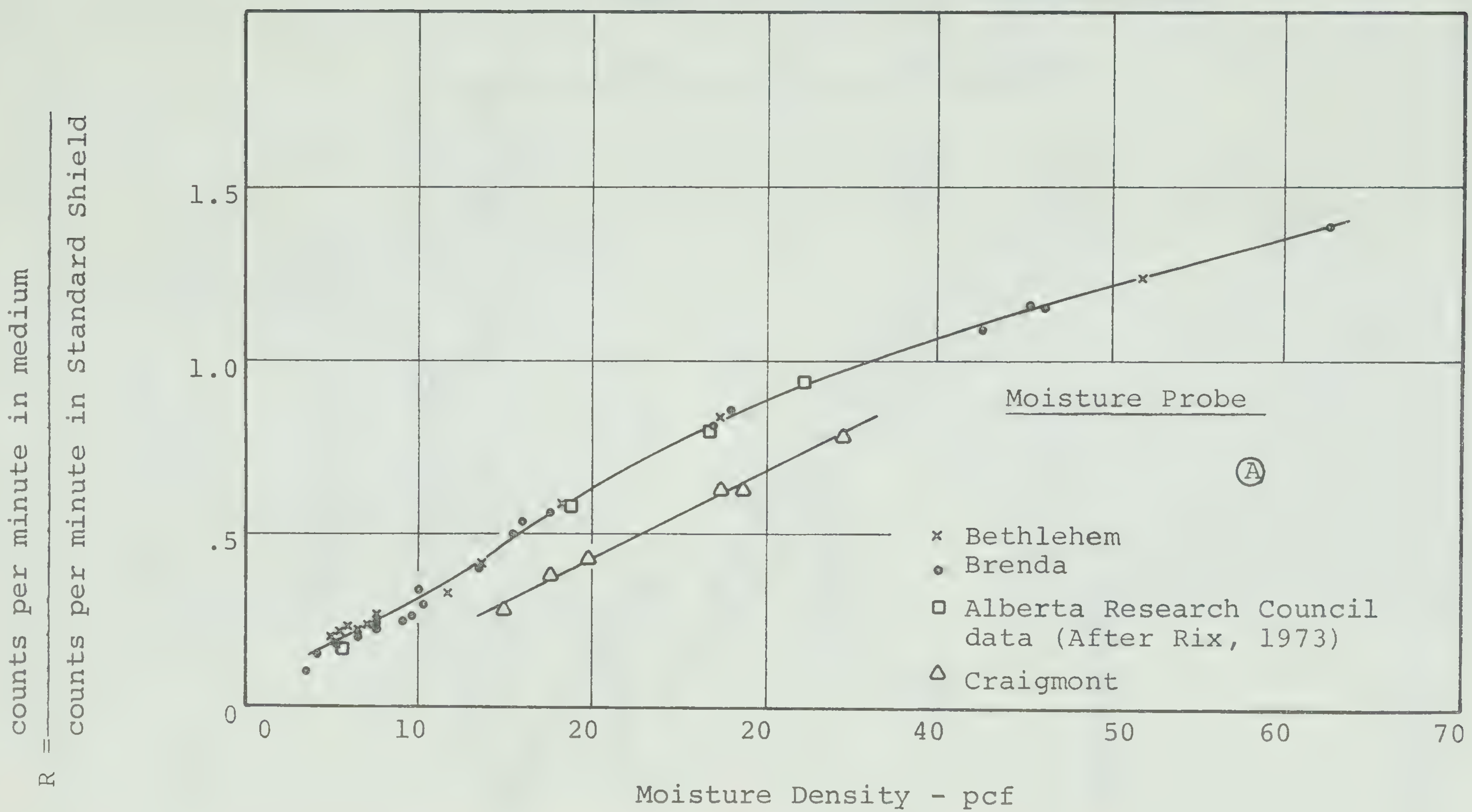


Figure 4.16 Nuclear Probe Calibrations

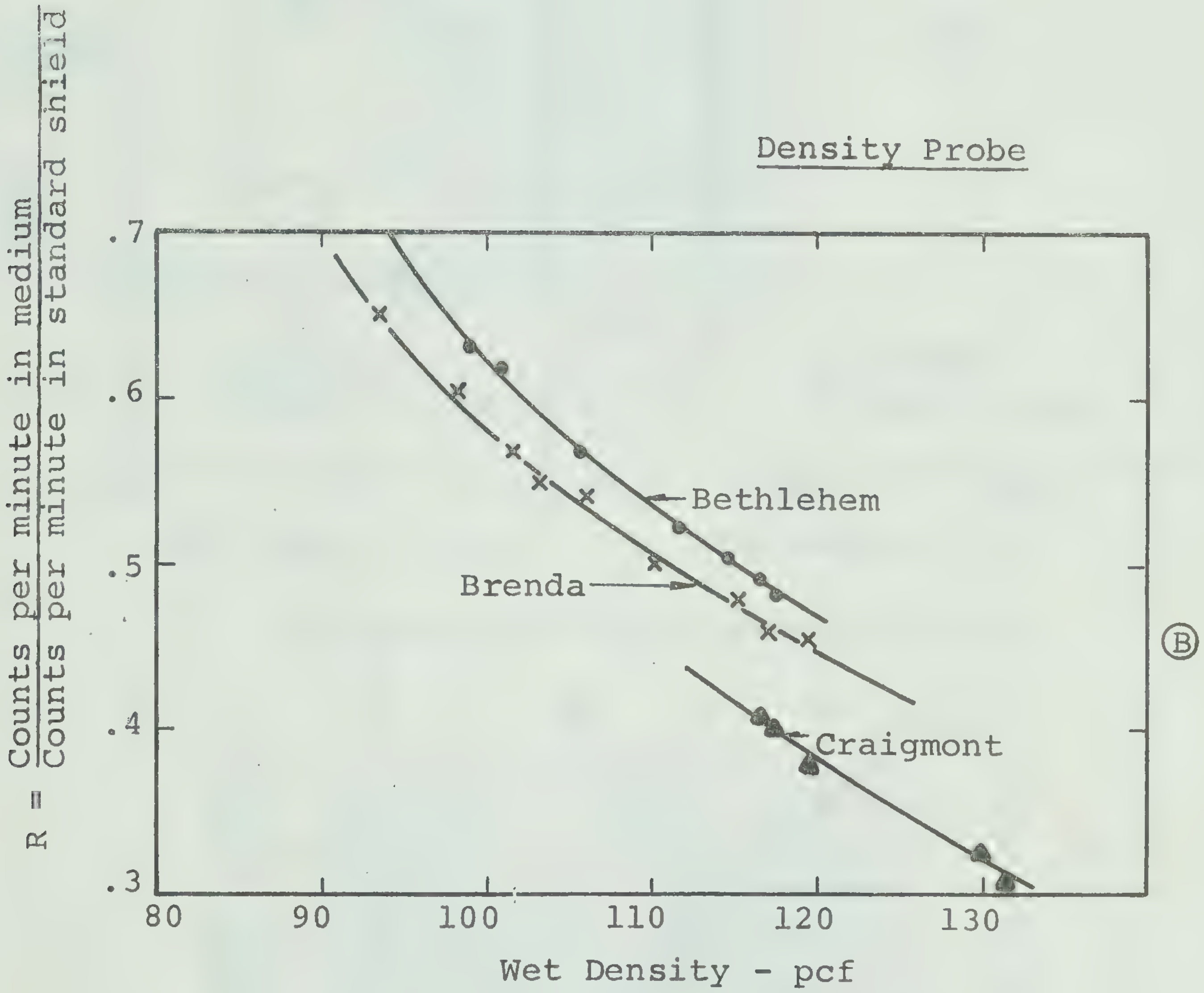


Figure 4.16 (cont'd)

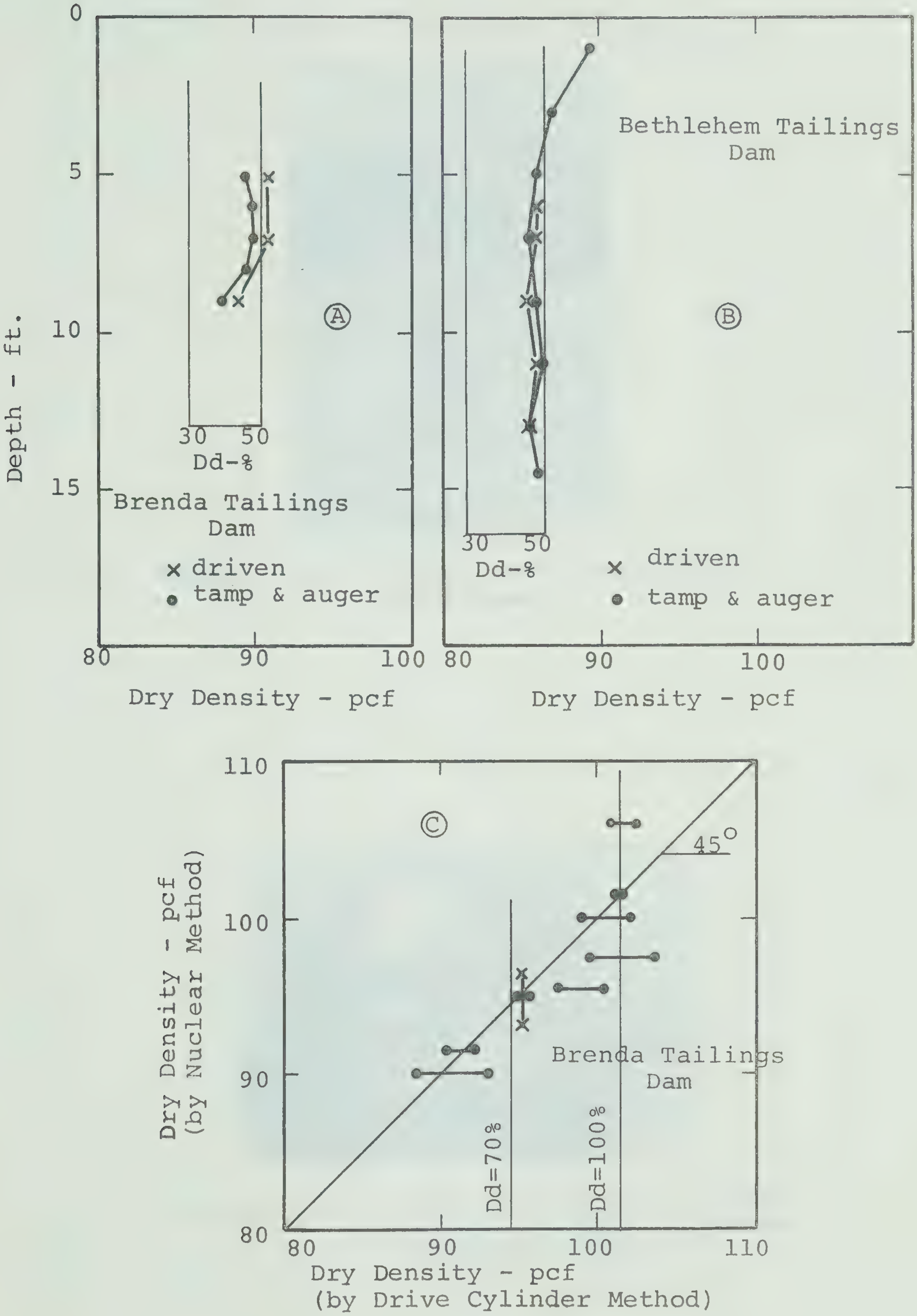


Figure 4.18 Comparative Density Tests

Figure 4.19 Tripod and drop hammer arrangement

Figure 4.20 A density sample by drive cylinder method inside the protective casing.

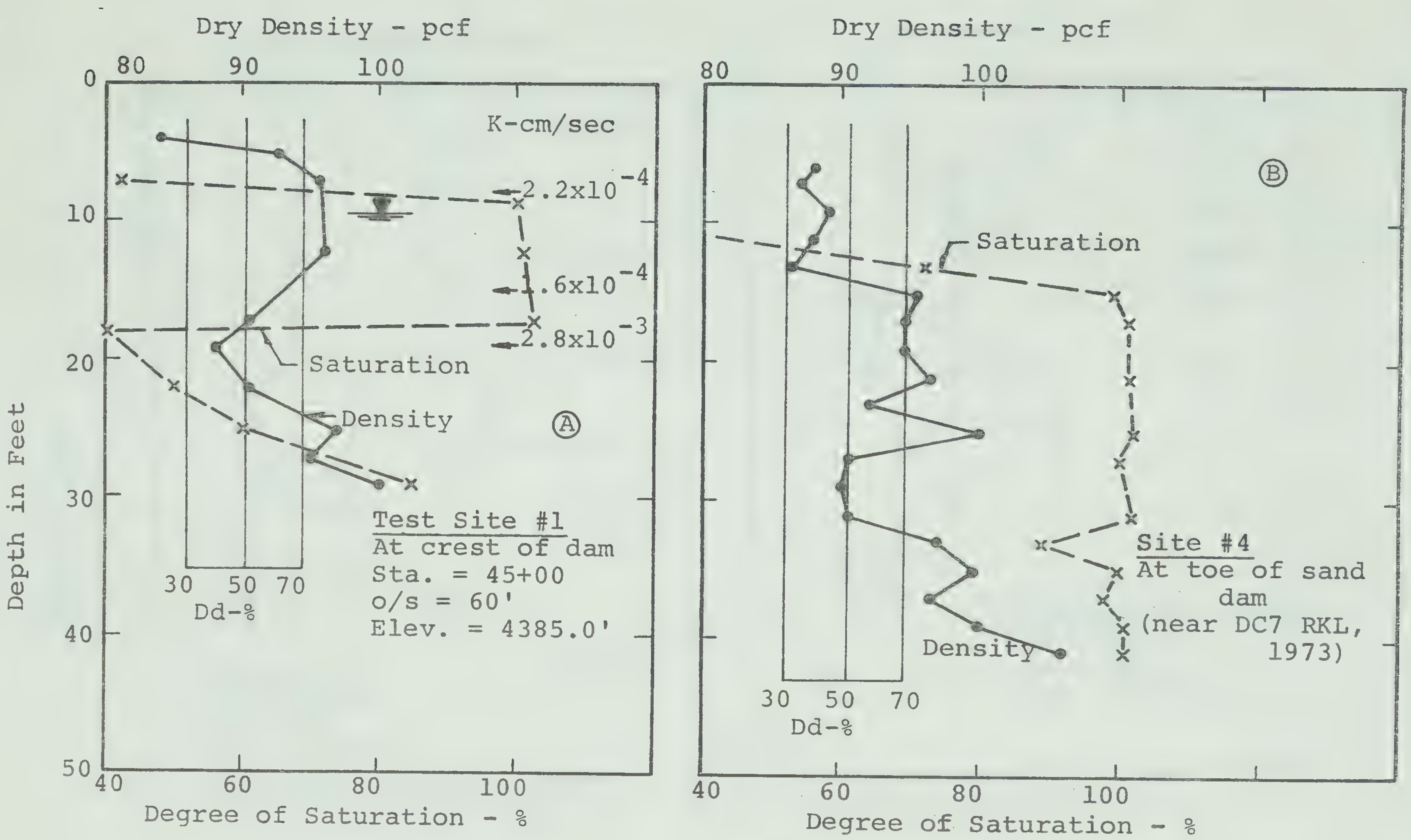


Figure 4.21 Typical Test Results - Brenda Site Where Perched Water Table Encountered

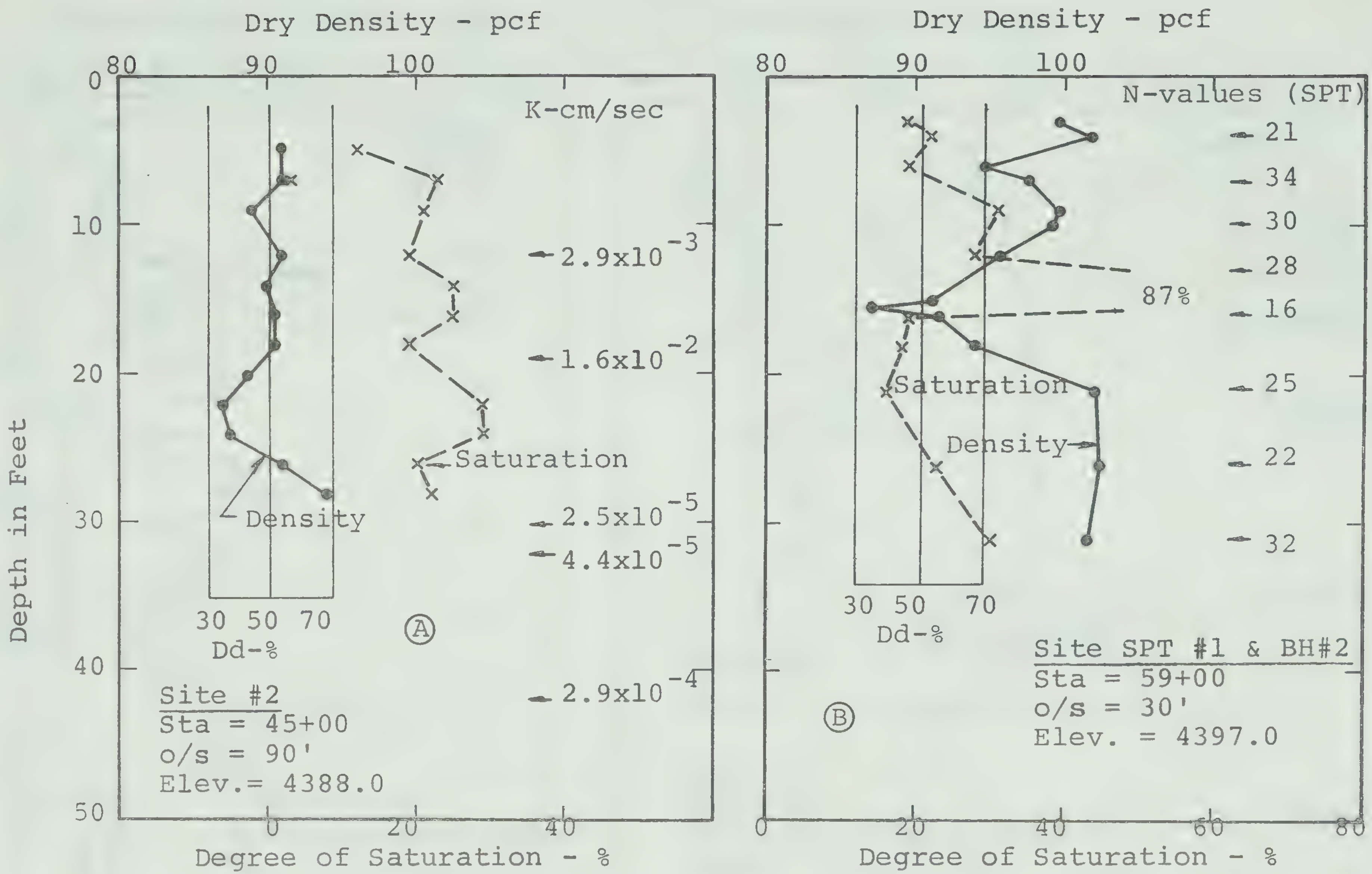


Figure 4.22 - Typical Density & Permeability Results Brenda Tailings Dam

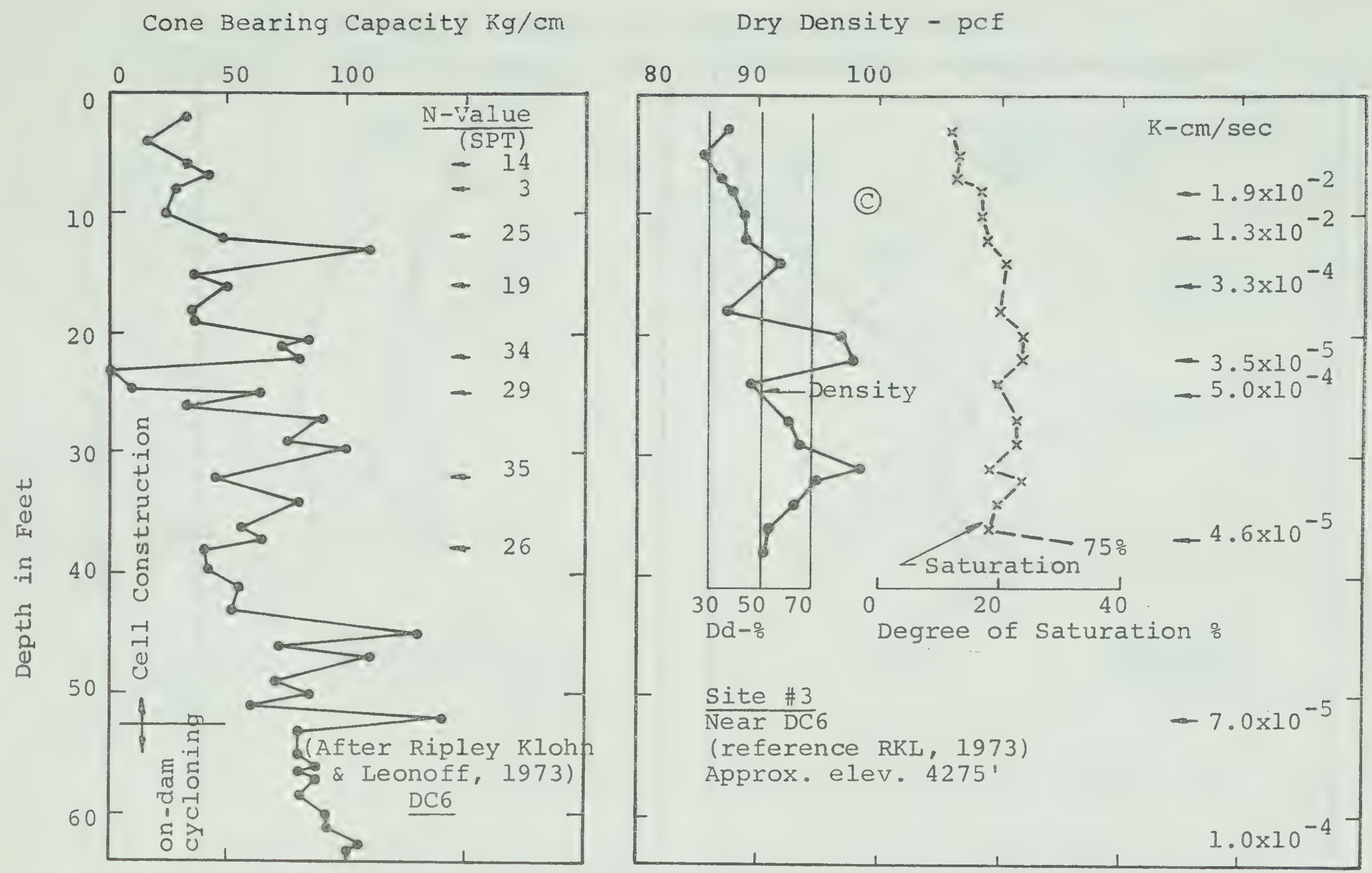


Figure 4.22 (cont'd)

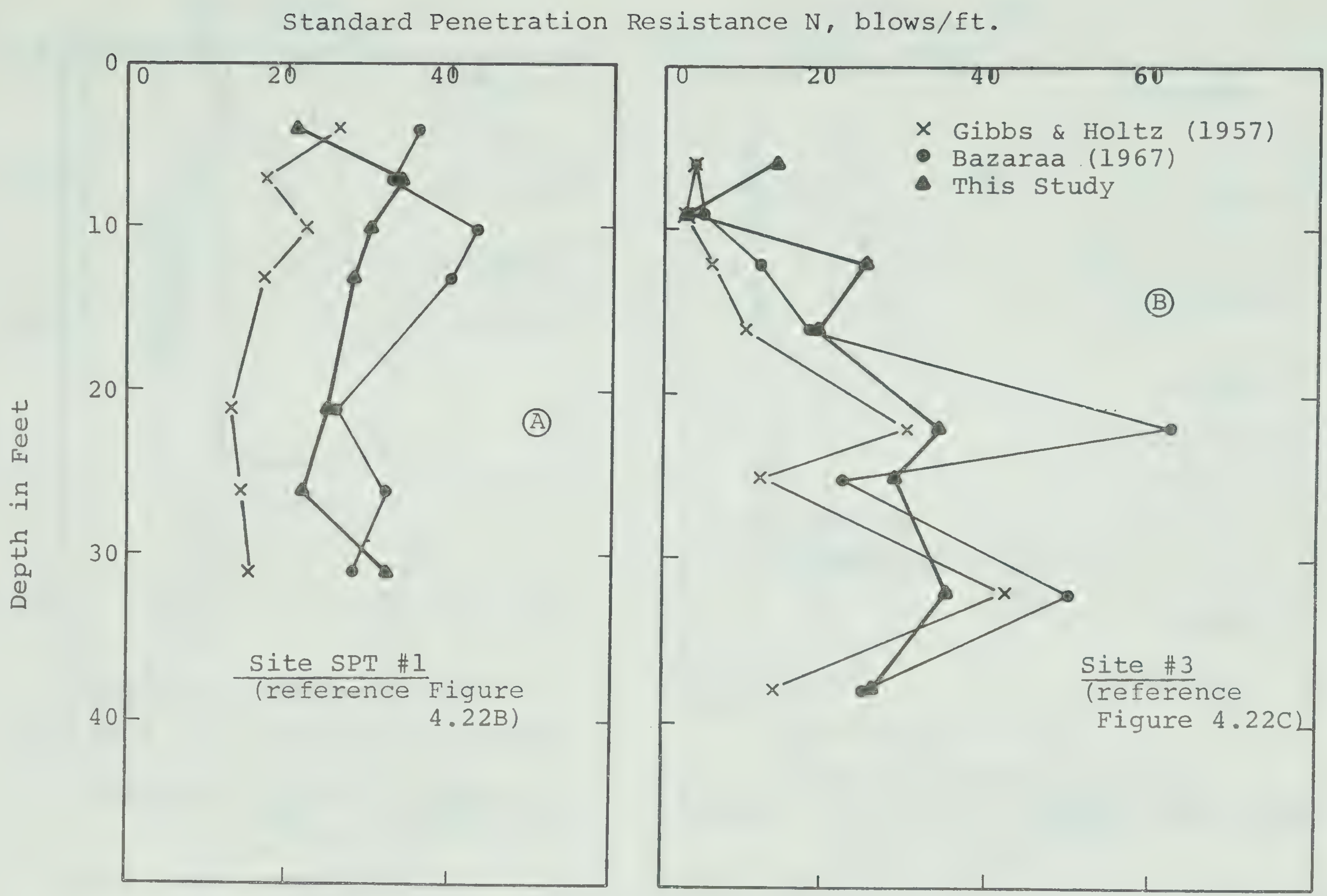


Figure 4.23 Comparative N - values by Different Methods (Reference Figure 4.22)

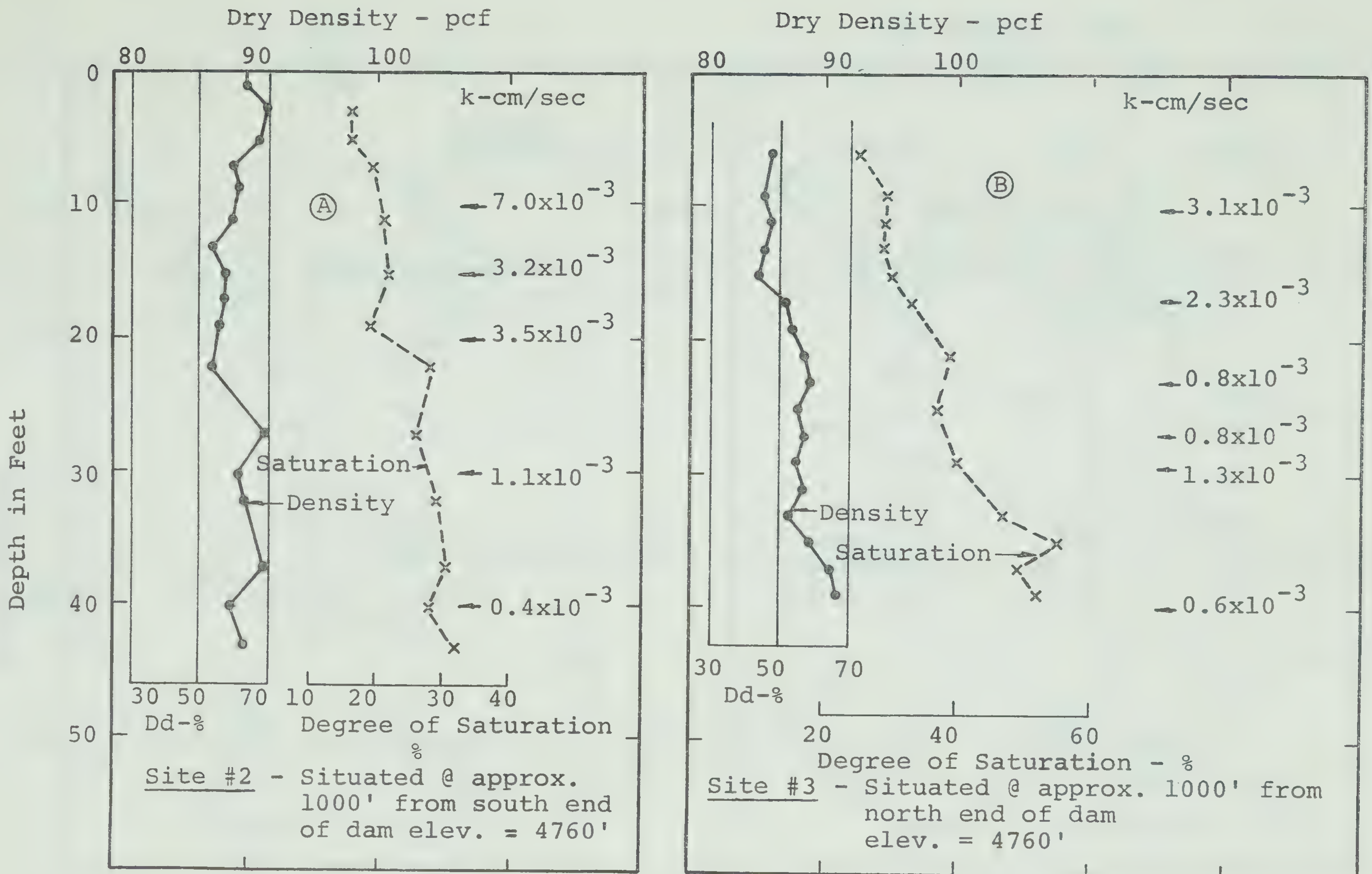


Figure 4.24 - Typical Density & Perm. Results Bethlehem Tailings Dam

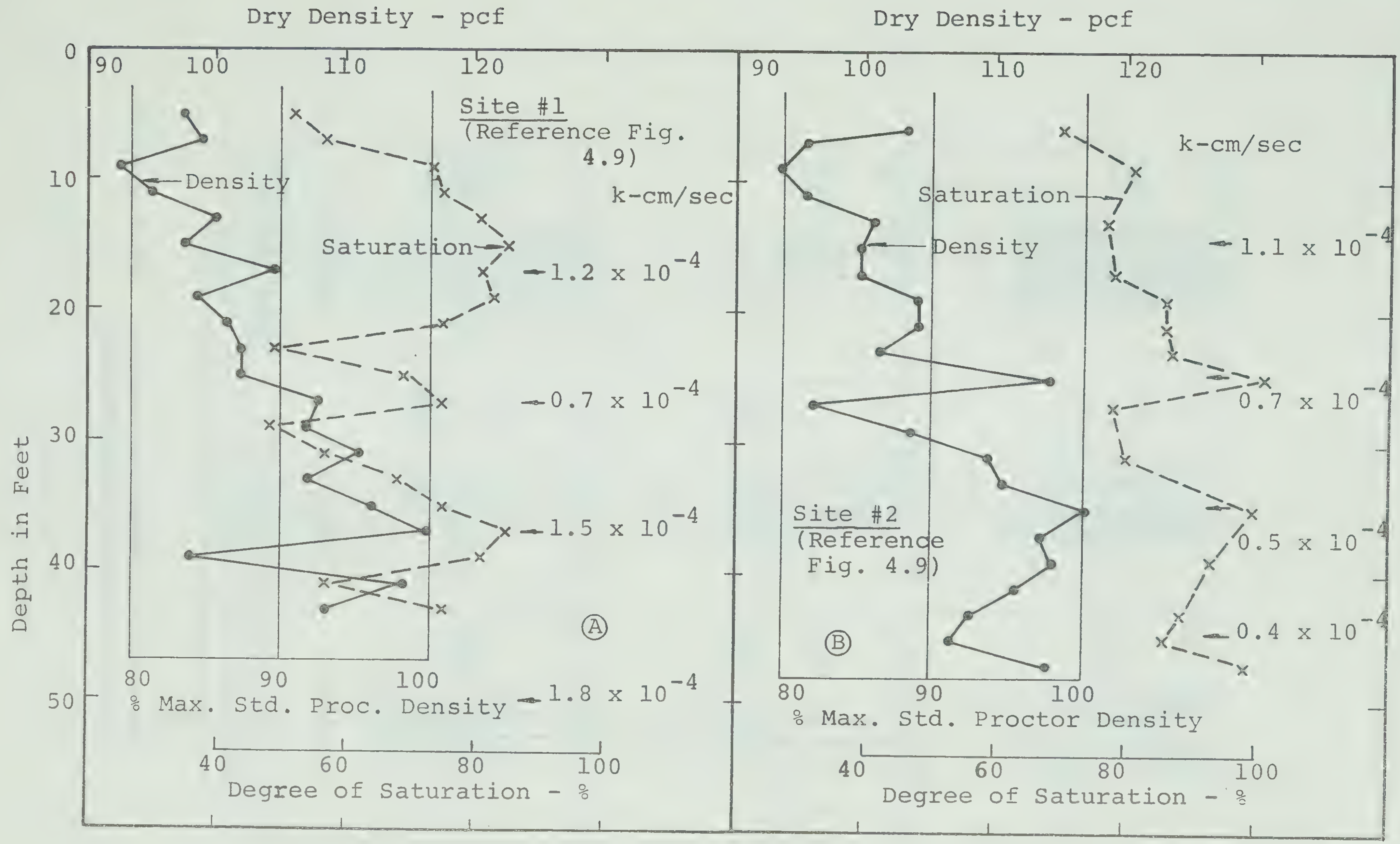


Figure 4.25 - Typical Density & Perm. Results
Craigmont Tailings Dam

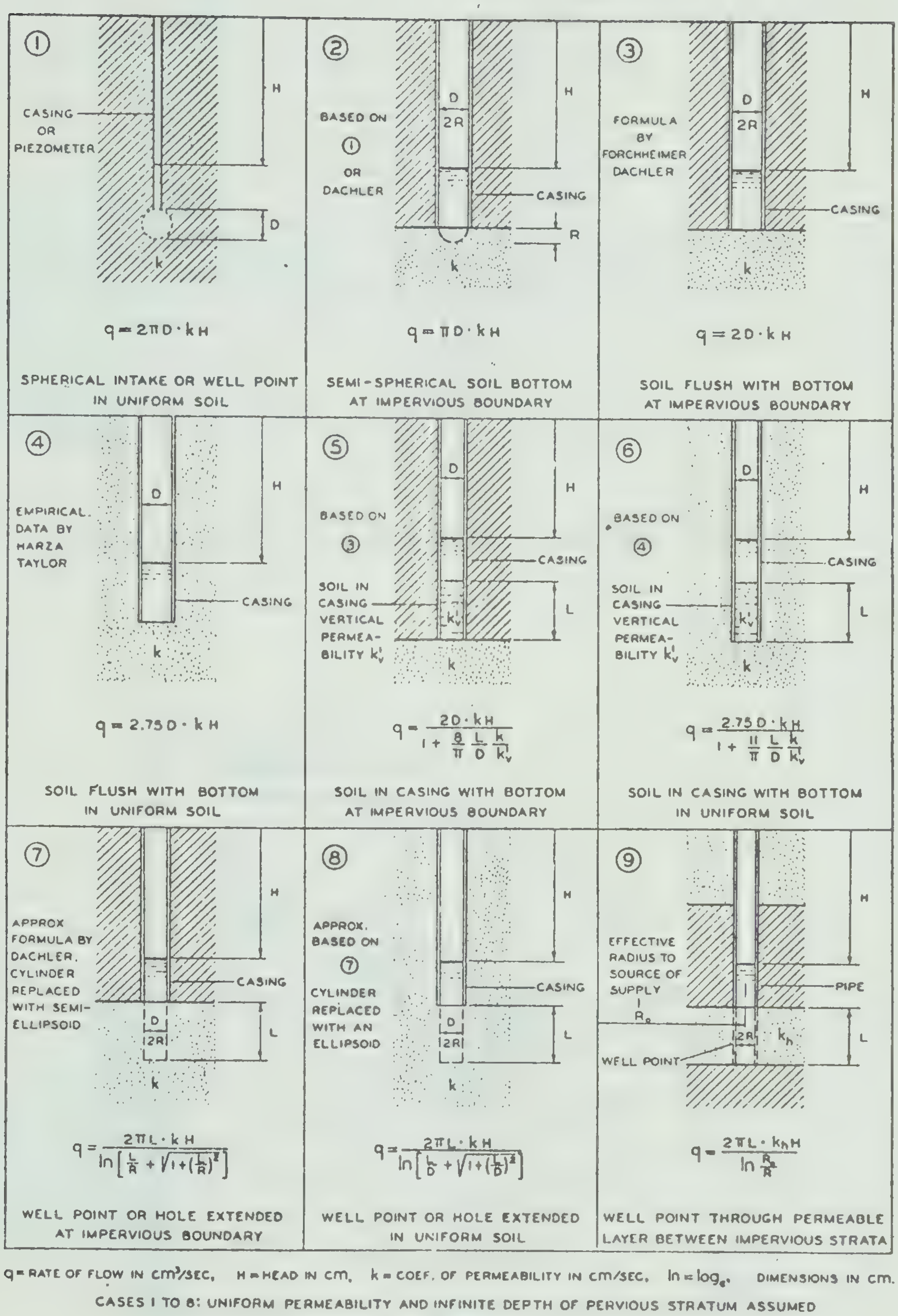


Figure 4.26 Inflow and Shape Factors for Piezometers (Hvorslev, 1951)

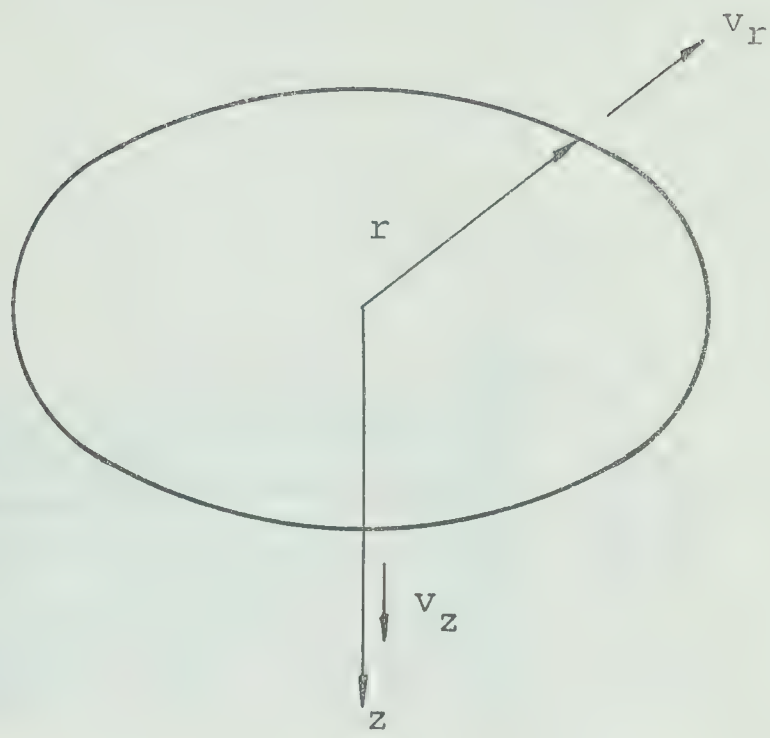
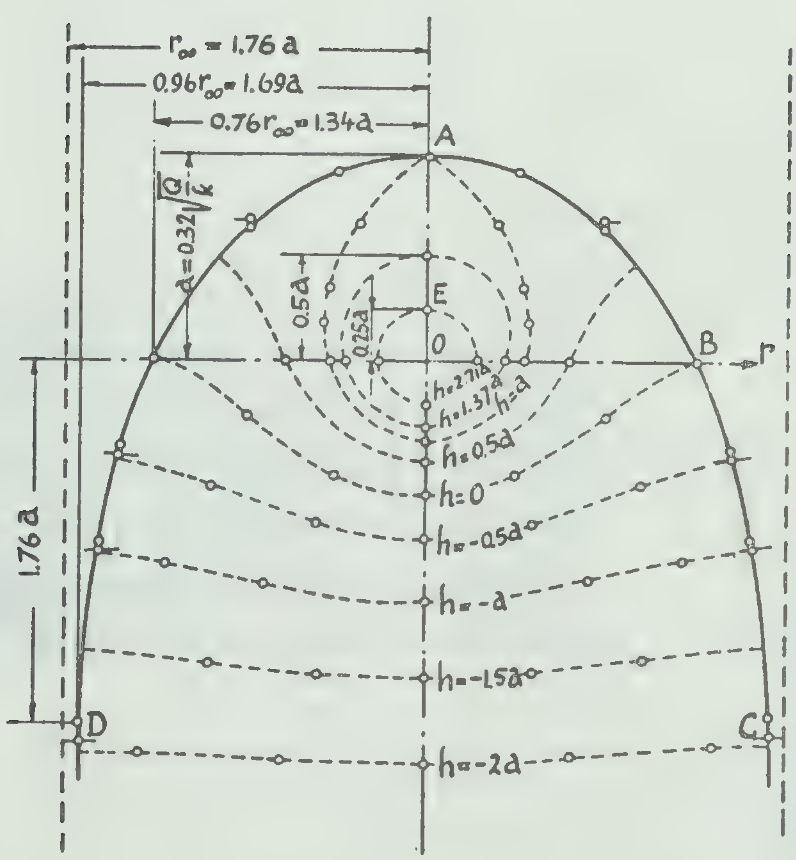
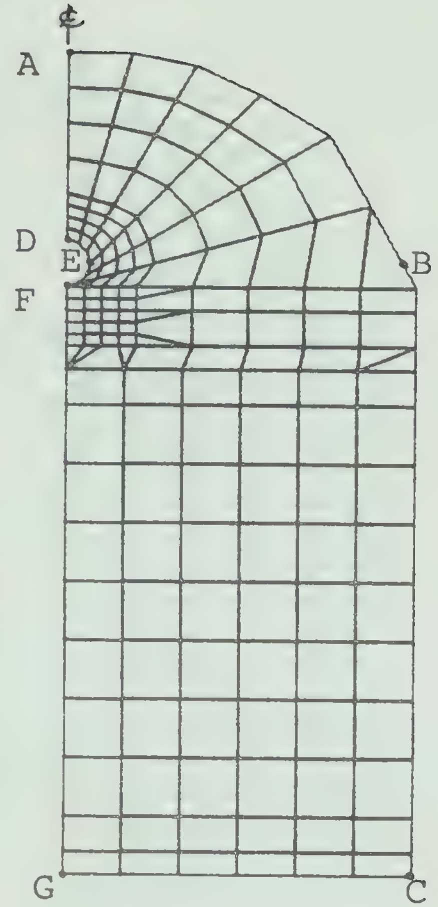


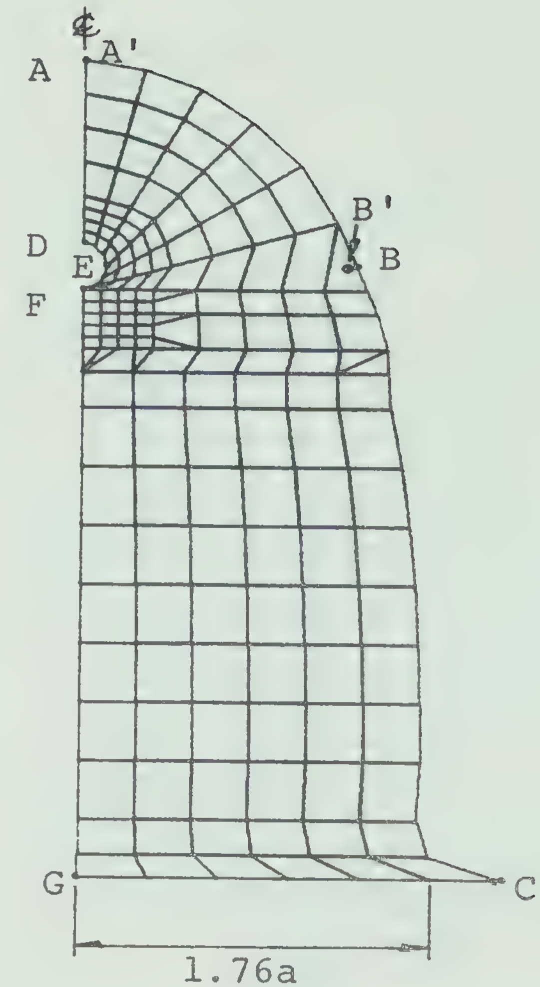
Figure 4.27 Coordinate System



(After Palubarinona - Kochina, 1962)
Figure 4.28 Seepage Flow From Point Source in Otherwise Dry Soil



A - Input

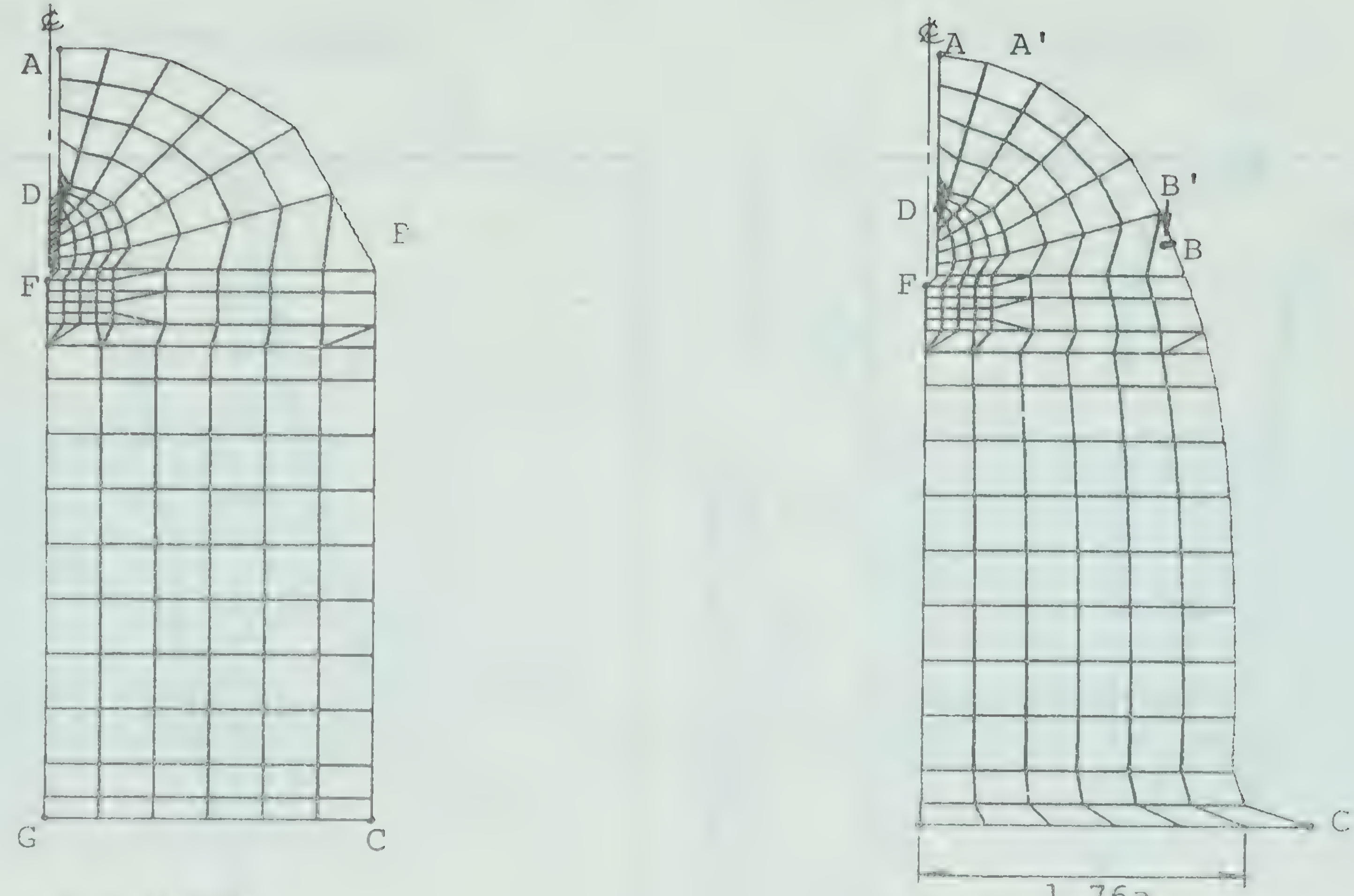


B - Output

Boundary Conditions

Boundary DEF - applied upstream equipotential
ABC - phreatic surface
AD & FG - impervious
CG - wetting front - zero equipotential line - pressure = 0

Figure 4.29 Finite Element Analysis for a Spherical Piezometer



A - Input

B - Output

Boundary Conditions
Same as in Figure 4.29

Figure 4.30 Finite Element Analysis for a Cylindrical Piezometer (L/D = 3)

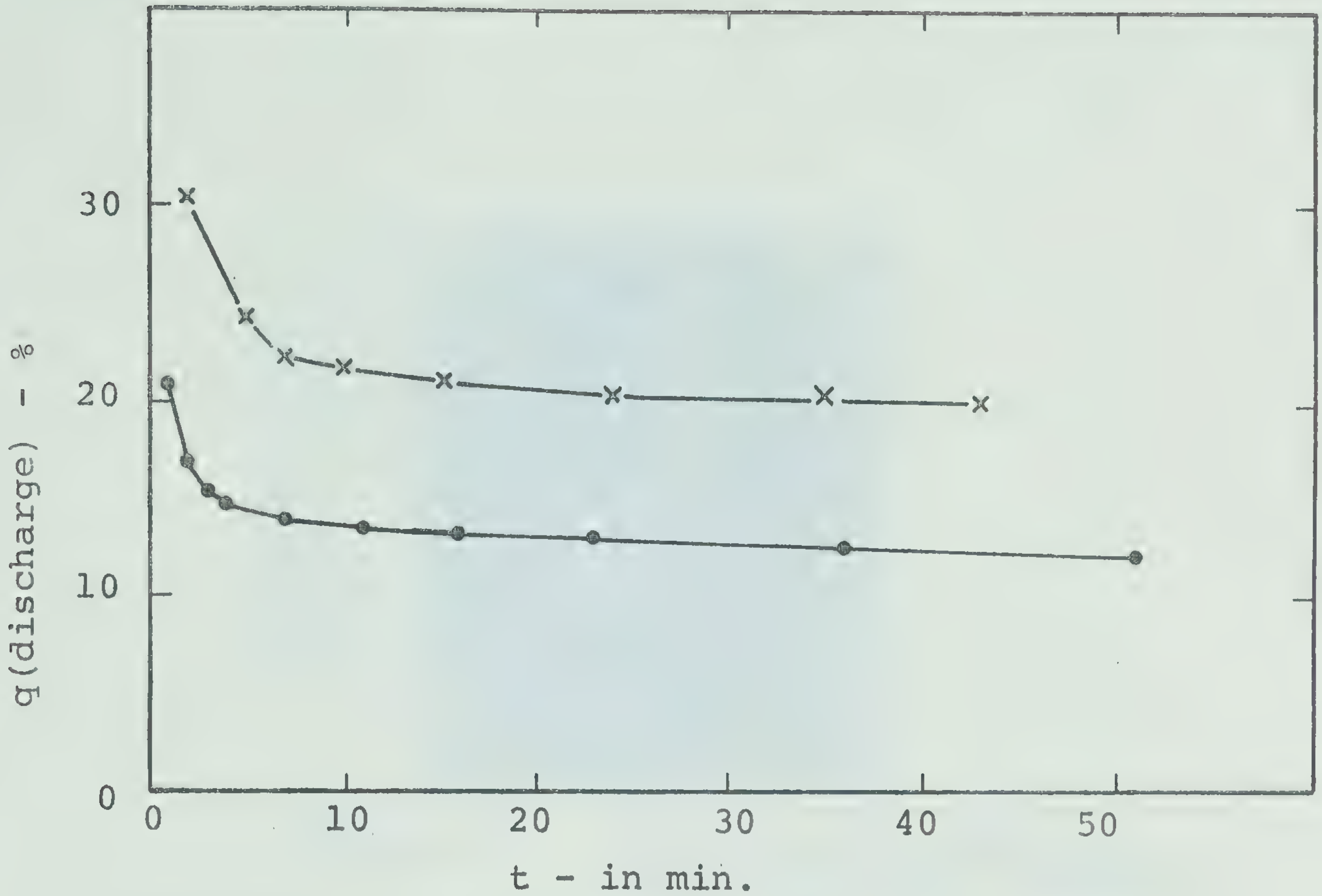


Figure 4.31 Typical Discharge Versus Time Plots from Model Piezometer Studies

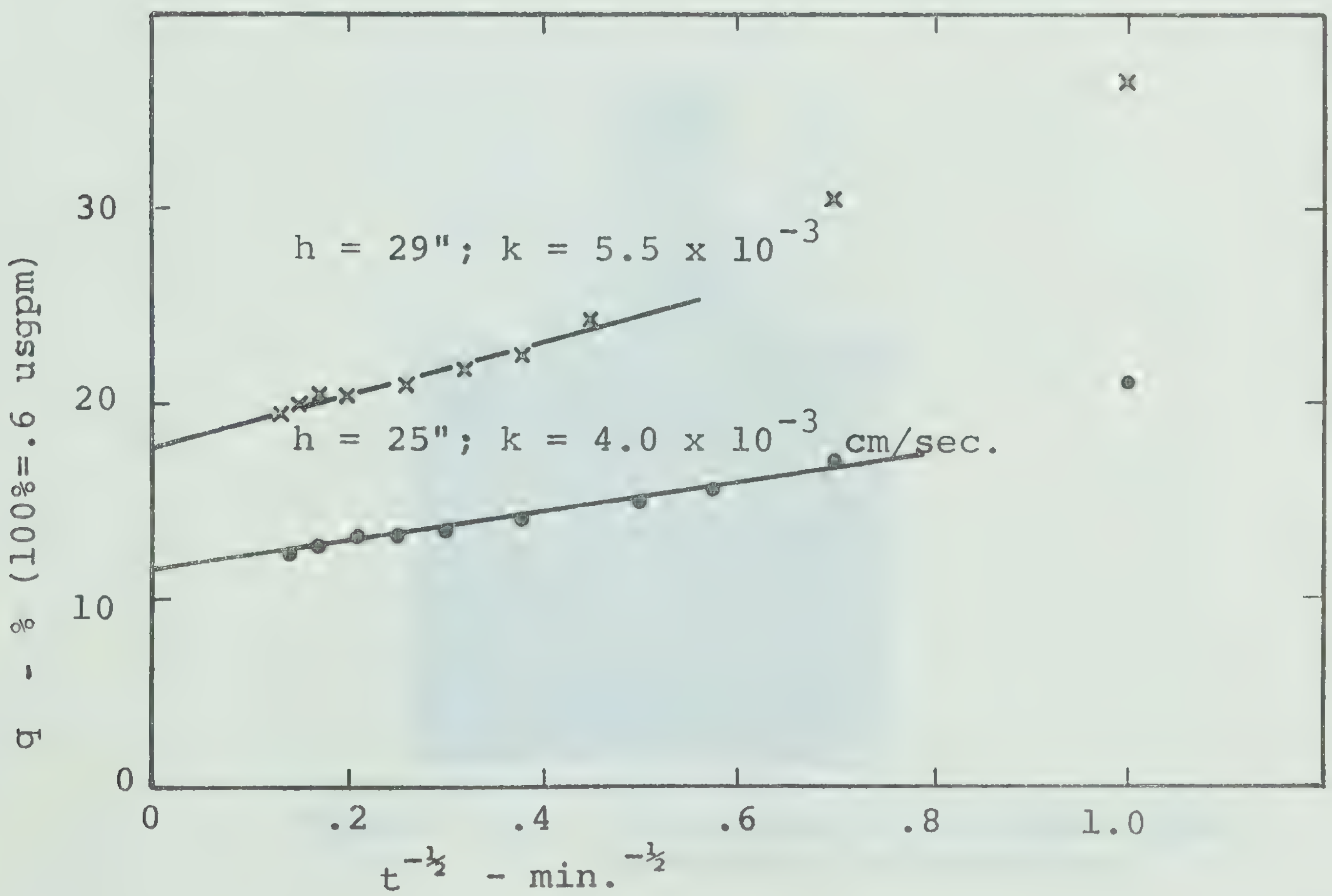


Figure 4.32 q Versus $t^{-\frac{1}{2}}$ Plots (Based on Figure 4.31)

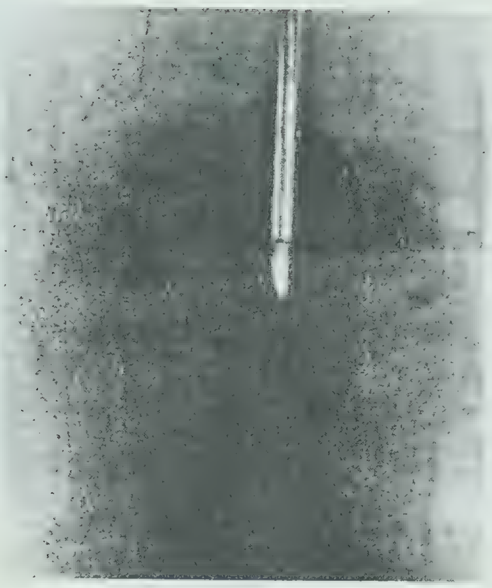


Figure 4.33 Photograph of wetted zone around the model piezometer

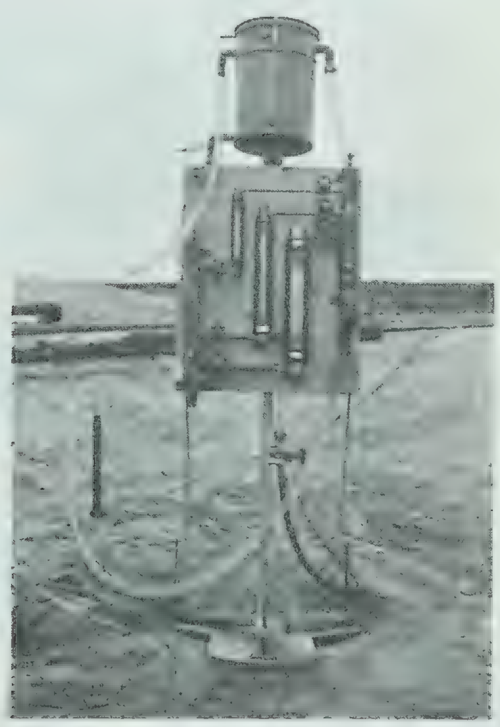


Figure 4.34 Photograph of constant head permeability apparatus

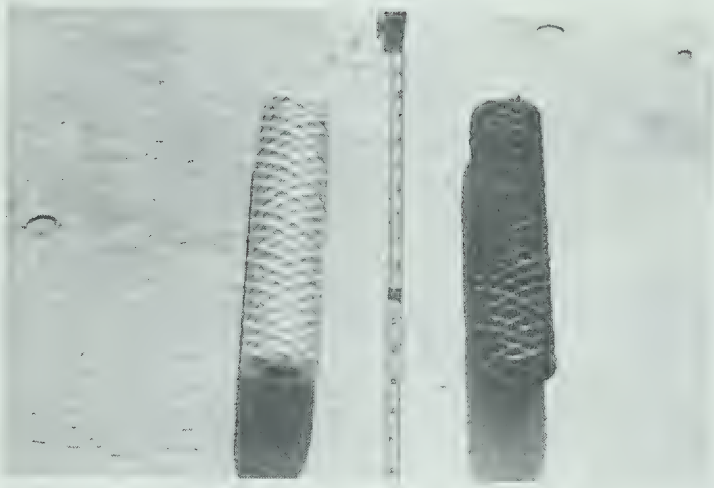


Figure 4.35 Photograph showing a 5 micron filter (before and after test)

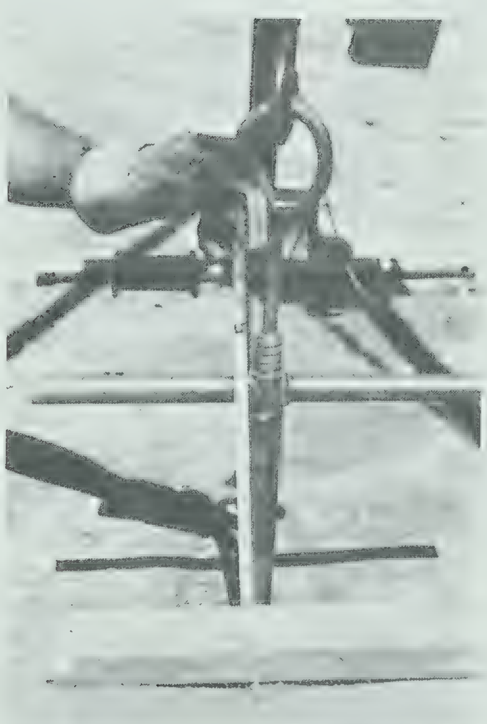


Figure 4.36 Photograph showing piezometer rods being pulled up to withdraw porous element from sleeve

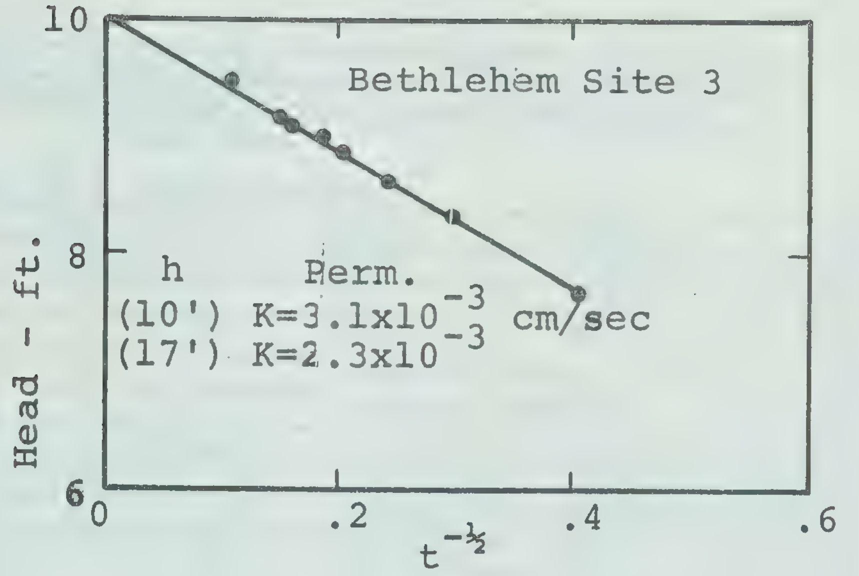
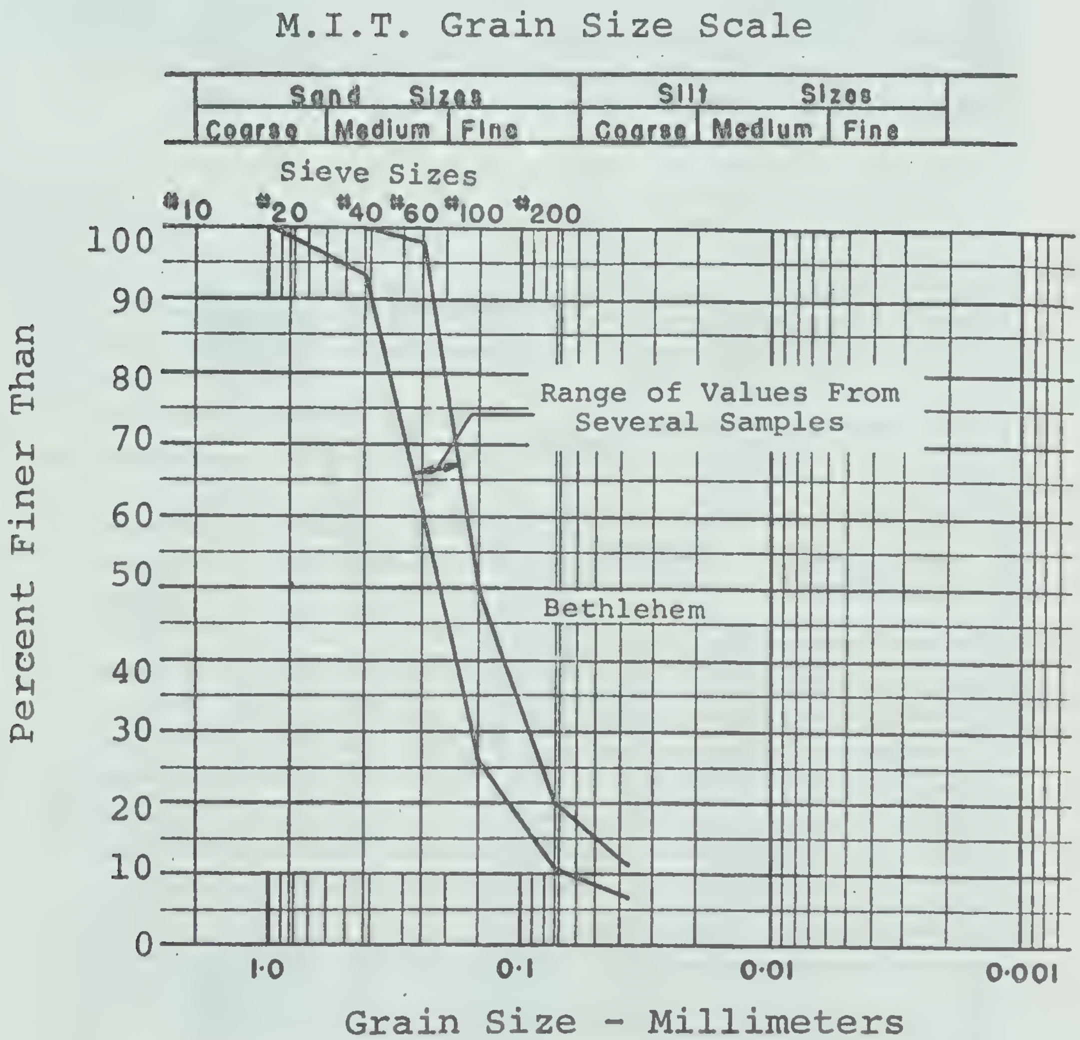


Figure 4.37 Typical Head Versus $t^{-1/2}$ Plot (Constant Discharge Test)



4.38 Grain Size Ranges- Field Samples

M.I.T. Grain Size Scale

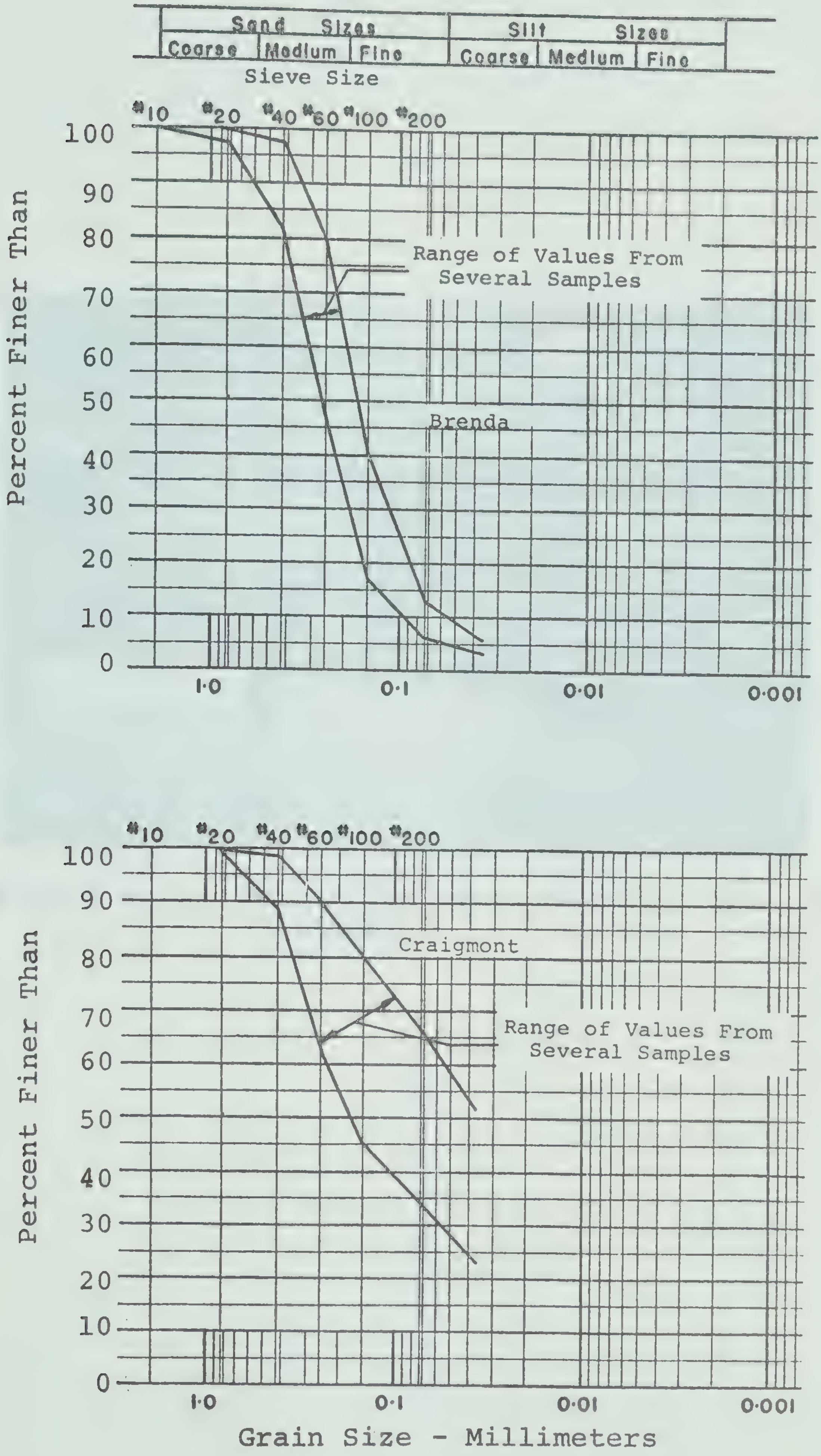


Figure 4.38 (cont'd)

Figure 4.39 Set-up for measurement of pore pressures in slimes.

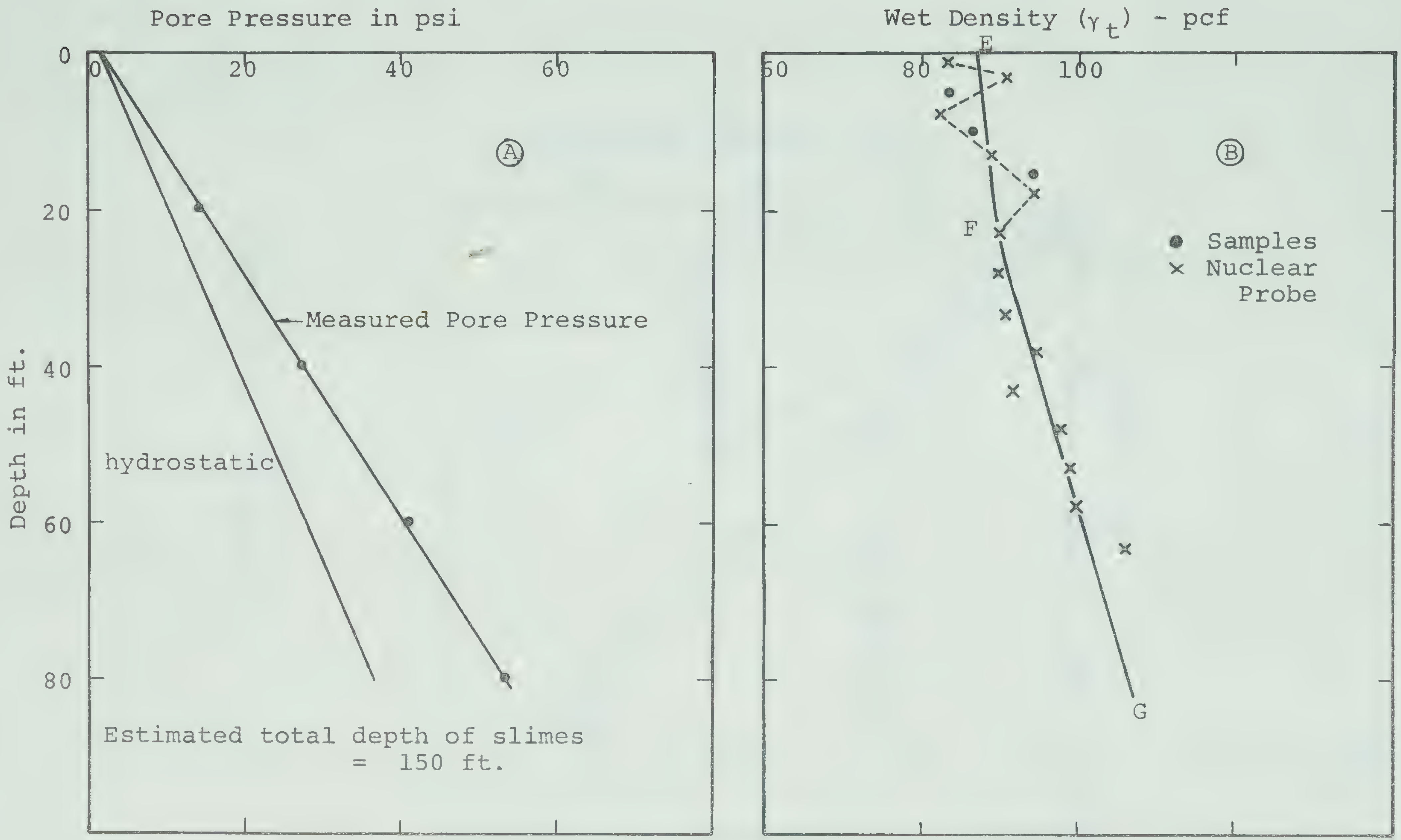


Figure 4.40 Pore Pressure and Density Results
Bethlehem Tailings Pond

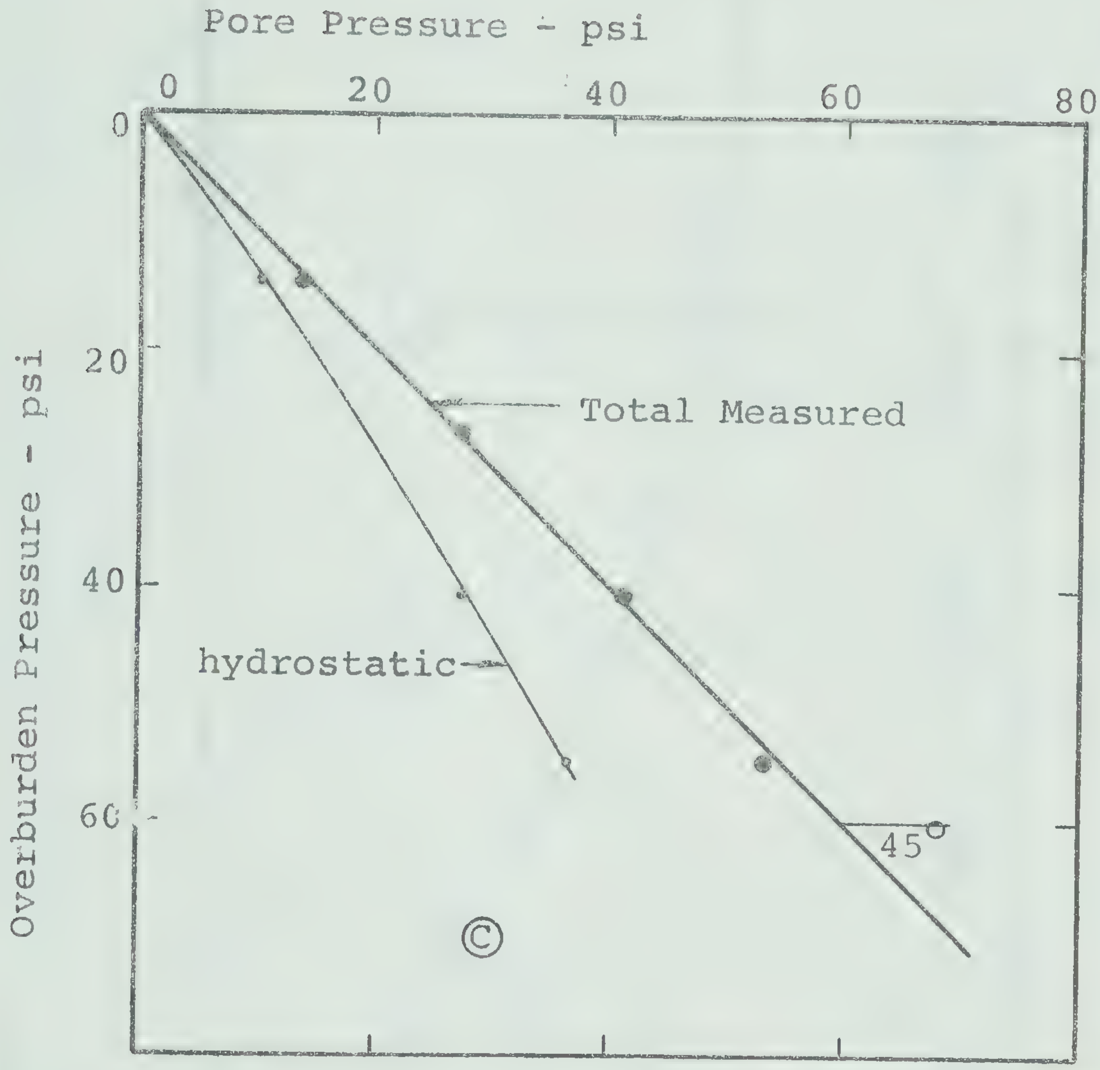


Figure 4.40 (cont'd)

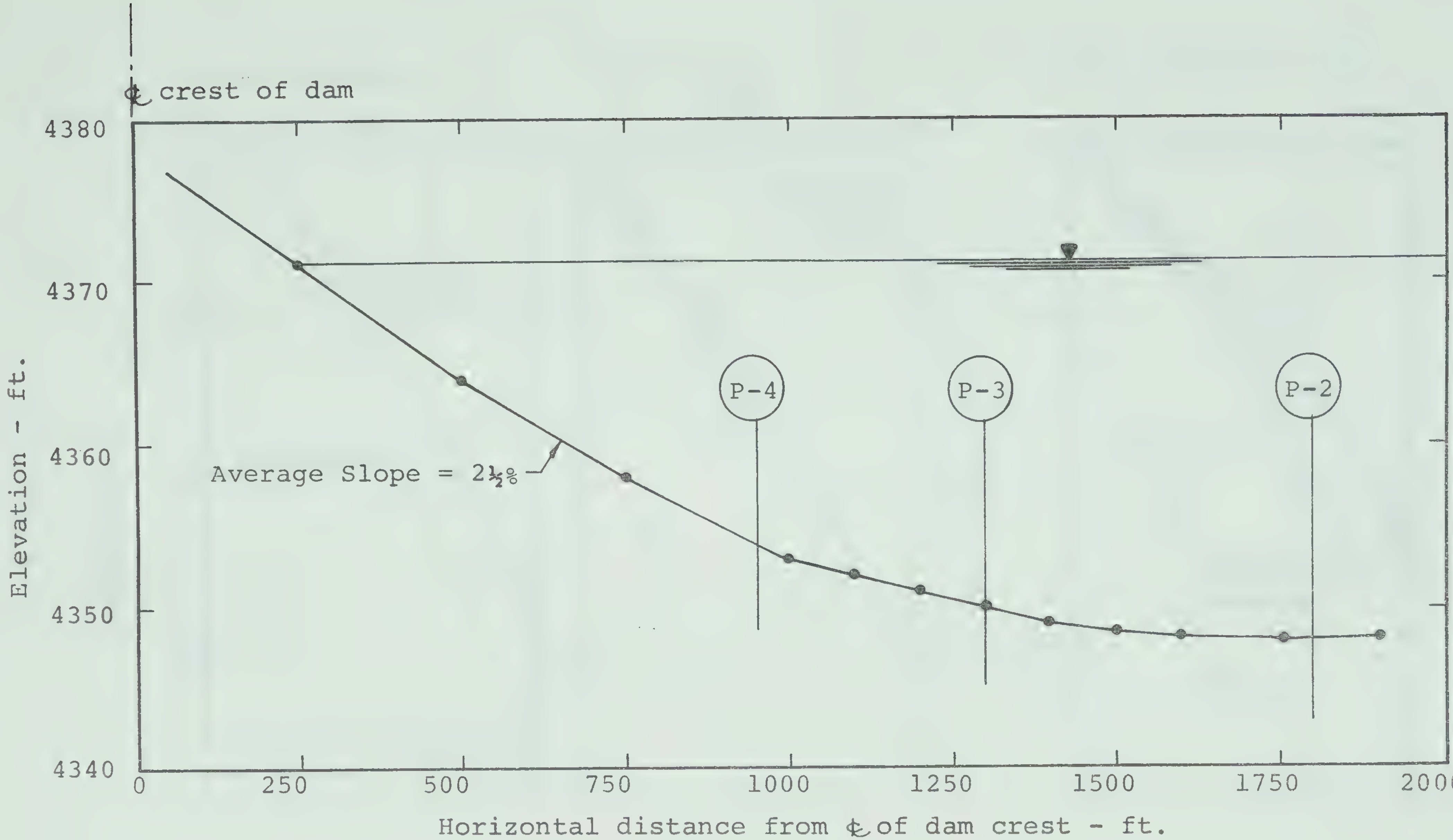


Figure 4.41 Section @ Sta. 55+00 showing test locations
Brenda Tailings Pond

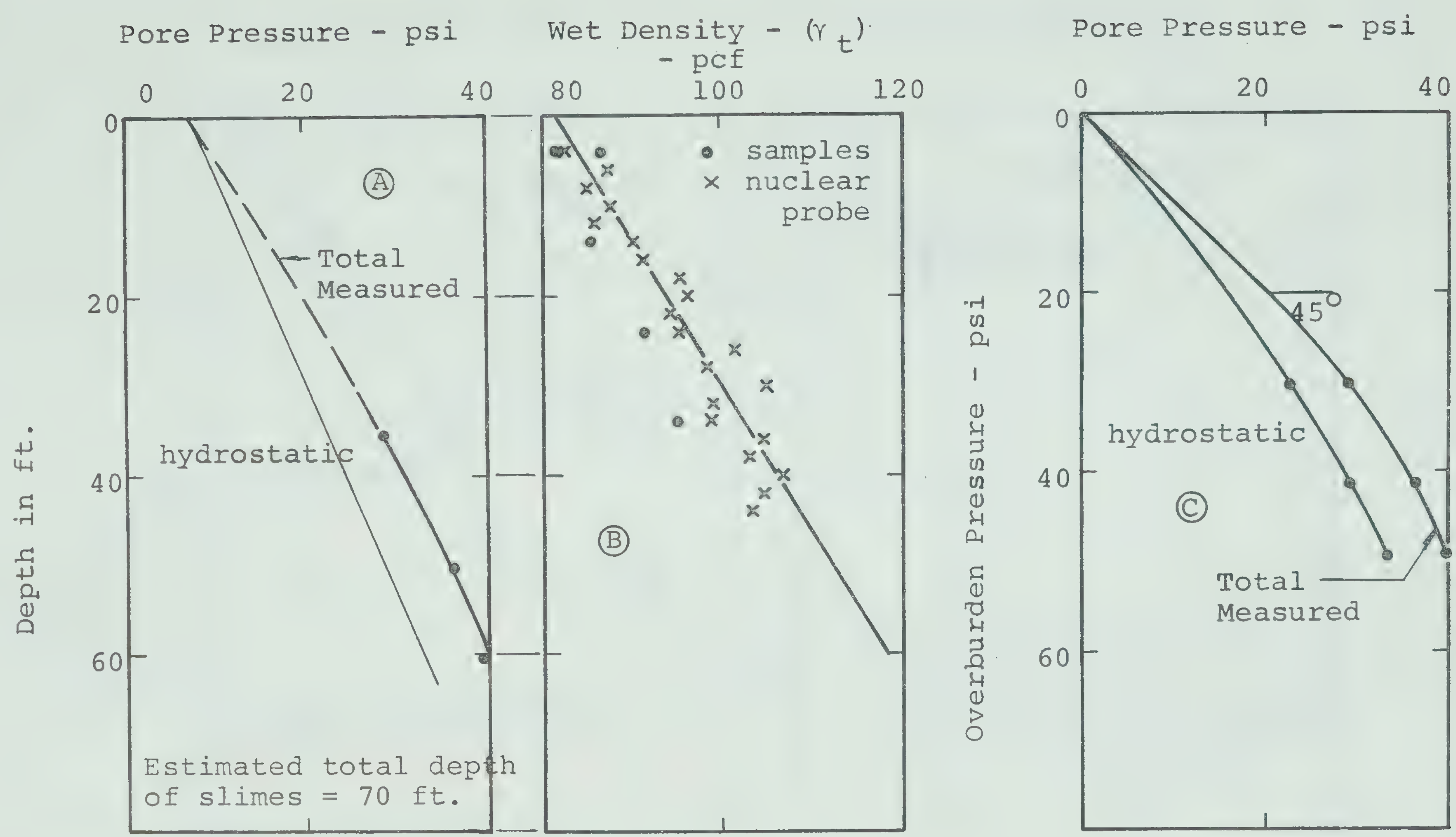


Figure 4.42 Pore Pressure and Density Results Brenda Tailings Pond - Test Location P-2

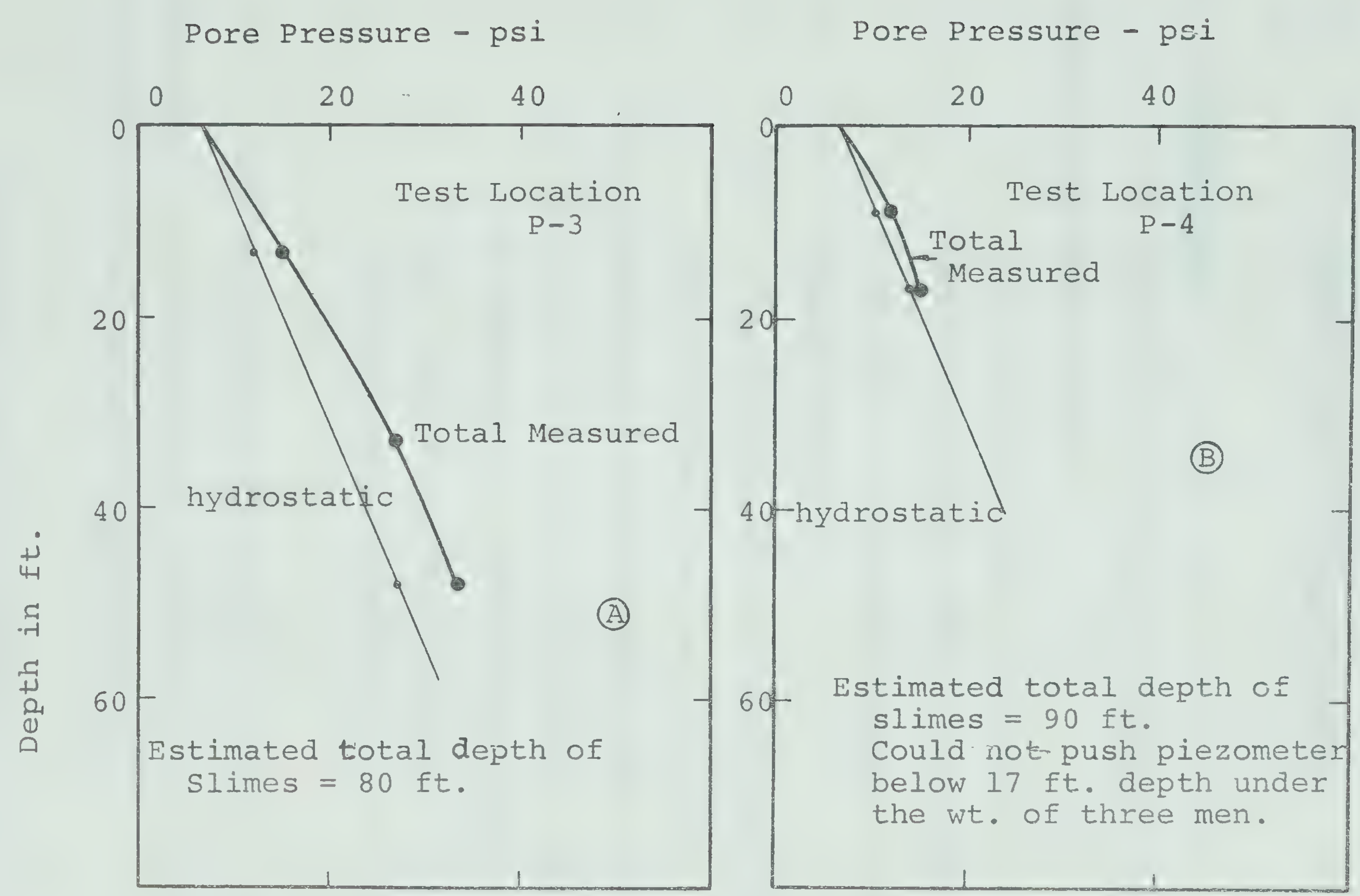


Figure 4.43 Pore Pressure Results
Brenda Tailings Pond
Test Locations P-3 & P-4

CHAPTER V

SEEPAGE THROUGH TAILINGS DAMS

5.1 Introduction

The following two items of seepage through tailings dams are considered here:

i) At the present time, the rate of seepage through a tailings dam is estimated from a flow net construction assuming a steady state seepage condition.

ii) In the construction of a tailings dam by downstream methods, an impervious seal is generally employed against the upstream face of the dam to minimize seepage flow through an embankment. Various techniques of employing this seal have been described in the literature as discussed in Section 2.4.3.5 of Chapter II.

The validity of the above methods has been questioned in a general sense in Chapter II; a detailed discussion on the subject is presented here.

It should be noted with respect to item (i) above that some relatively low tailings dams are built at sufficiently low rates of construction for complete dissipation of

excess pore pressures. Seepage flow through these dams can be adequately represented by the conventional techniques applicable to steady state condition (Kealy and Busch, 1971) and is not an issue here. Where pond levels rise at rather rapid rates, however, slimes stored behind the dams remain in an under-consolidated condition with excess pore pressures. Seepage flow through tailings dams under these conditions is the subject discussed in this chapter. Furthermore, to simplify the problem for presentation purposes, discussion will be limited to the tailings dams constructed by the downstream methods only, although some of the comments made here are equally applicable to dams constructed by other methods.

5.2 Seepage Due to Consolidation of Slimes

5.2.1 Overview

After initial sedimentation of solids into a very loose soil, the slimes material is subjected to the long term process of consolidation. In the central portion of a tailings pond, the process is of only one-dimensional consolidation, i.e. due to drainage of pore water from the slimes in the vertical direction only. Whereas in the material immediately upstream of the sand dam, consolidation is of two-dimensional type, i.e. due to pore water drainage vertically as well as horizontally through the sand dam. It is shown on the basis of the work of Gibson (1958) that in the slimes immediately upstream of a dam, dissipation of

excess pore pressures due to vertical drainage is rather small as compared to that which occurs due to horizontal drainage. Therefore, for all intents and purposes, consolidation in this zone is also essentially of one-dimensional nature but in this case due to horizontal drainage through the dam. It is hypothesized that seepage through a tailings dam is due to this horizontal drainage from the consolidation of slimes.

5.2.2 Consolidation of Slimes in a Tailings Pond

The results of in situ pore pressure measurements presented in the previous chapter indicate that the slimes contain high excess pore pressures beyond a certain distance from the sand dam. It has been also indicated previously that the progress of consolidation in the slimes can be assessed by analytical methods presented by Gibson (1958). A brief description of Gibson's presentation follows.

Within the framework of the usual assumptions made in the theory of one-dimensional consolidation, Gibson (1958) presents an analytical solution for estimating progress of consolidation in a clay layer which is increasing in thickness with time. The problem as shown in Figure 5.1 has been considered where $h(t)$ and $H(t)$ are specified functions of the elasped time t since deposition commenced, and the initial thickness $h(o)$ of the layer is taken as zero. Numerical values of excess pore water pressure u have been

evaluated for the following two different rates of deposition:

i) constant rate of deposition, i.e., $h = mt$

ii) thickness of deposit proportional to $t^{\frac{1}{2}}$, i.e., $h = \lambda t^{\frac{1}{2}}$.

Gibson suggests that for any other rate of deposition, values of excess pore pressures can be computed by numerical integration of the governing equation. Results for the above two rates of deposition are presented in Figures 5.2 and 5.3; a brief discussion follows.

For an impervious base, the results are shown plotted in Figure 5.2 in terms of $u/\gamma' h$ versus x/h for various values of $\lambda/2\sqrt{C_v}$ and $m\sqrt{t}/2\sqrt{C_v}$; where $\gamma' = \gamma_t - \gamma_w$ and u is excess pore pressure. Similar results are presented in Figure 5.3 for a case of pervious base but in terms of $P_w/\gamma_t h$ versus x/h where P_w is the total pore-water pressure and H(t) is equal to h(t).

A review of Figures 5.2A and 5.3A where rate of deposition is proportional to $t^{\frac{1}{2}}$ indicates that the distribution of excess pore pressure through the thickness of the deposit is controlled only by the time independent parameter $\lambda/2\sqrt{C_v}$ and it follows therefore, that the average degree of consolidation of the layer is independent of time. For the constant rate of deposition, however, the distribution of excess pore pressures is controlled by the time

dependent parameter $m\sqrt{t}/2\sqrt{C_v}$ and the average degree of consolidation of the layer decreases with time giving its lowest value at the end of deposition.

It is of interest to note from Figure 5.2A and 5.2B that at any given stage during deposition of slimes, equivalent values of parameters $\lambda/2\sqrt{C_v}$ and $m\sqrt{t}/2\sqrt{C_v}$ produce almost similar distributions of excess pore pressures where m represents an average rate of deposition to that point since deposition commenced. The same can be said about Figures 5.3A and 5.3B. Therefore, if λ is equal to $m\sqrt{t}$ as illustrated in Figure 5.4, the distribution of excess pore pressures can be reasonably assessed by assuming a constant rate of deposition for a deposit which in fact follows the relationship $h = \lambda t^{\frac{1}{2}}$ or any other relationship between this and $h = mt$.

The pond levels have been shown plotted against time in Figure 5.5 for Bethlehem and Brenda tailings ponds. The average rates of deposition have been estimated in the period for which records are available as shown in the above figure.

The value of C_v for Bethlehem slimes at initial high void ratios is estimated from Figure 3.31 to be about 1×10^{-3} cm^2/sec or 34 ft^2/yr. At the test location in the pond, the estimated depth of slimes is 150 feet as

shown in Figure 4.40. It follows, therefore, that at a constant rate of deposition of 20 ft/yr (Figure 5.5B) t is equal to 7.5 years and parameter $m\sqrt{t}/2\sqrt{C_v}$ equal to a value of 4.7. It is obvious from Figure 5.2B that for this value of $m\sqrt{t}/2\sqrt{C_v}$, there is hardly any consolidation within the top 70 percent thickness of the deposit and very little below that. The results from field measurements as given in Figure 4.40 indicate no consolidation to the investigated depth of 80 feet which is equivalent to about 53 percent of the total thickness of slimes at the test location.

The value of C_v for Brenda slimes at initial high void ratios can be estimated to be about 5×10^{-3} cm^2/sec or 170 ft^2/yr from Figure 3.31. At test location P-2 in the pond, the estimated depth of slimes is 70 feet as shown in Figure 4.42. It follows, therefore, that at a constant rate of deposition of 30 ft/yr (Figure 5.5A) t is equal to 2.3 years and parameter $m\sqrt{t}/2\sqrt{C_v}$ equal to a value of about 1.8. For this value of parameter $m\sqrt{t}/2\sqrt{C_v}$ from Figure 5.2B, there is no consolidation within about the top 35 to 40 percent thickness of the deposit, and there is an increasing degree of consolidation below this level. The results from field measurements (Figure 4.42) indicate no consolidation to a depth of about 30 feet which is equivalent to 43 percent of the estimated thickness of slimes at the test location.

On the basis of the above discussions, it is

concluded that in the later stages of disposal of tailings, slimes in the central portions of the pond exist in an under-consolidated state with high excess pore pressures.

5.2.3 Consolidation of Slimes Near the Sand-Slime Interface

5.2.3.1 General

It has been indicated previously, that slimes in the central portions of a pond are subjected to one-dimensional consolidation with drainage in vertical direction only. Near the sand-slime interface, however, the materials are subjected to two-dimensional consolidation with drainage occurring both in horizontal and vertical directions. But dissipation of excess pore pressures in slimes at the interface can be treated as a case of one-dimensional consolidation due to horizontal drainage of pore water through the sand-slime interface.

5.2.3.2 The Governing Differential Equation

The following assumptions are made in this formulation in addition to those usually adopted in the theory of one-dimensional consolidation:

i) the slimes pond is semi-infinite in horizontal extent i.e. $0 \leq x \leq \infty$ (see Figure 5.6),

ii) there is no vertical drainage; all drainage is in the horizontal direction through the sand-slime interface at $x = 0$,

iii) the current thickness of $h(t)$ is a specified

function of time t since deposition commenced, and the initial thickness of the layer h(o) is equal to zero.

For a thin horizontal layer at a distance y from the base,

$$\sigma_y = (h-y)\ \gamma_t$$

or

$$\frac{\partial \sigma_y}{\partial t} = \gamma_t \frac{dh}{dt} \qquad \text{.....(5.1)}$$

and the basic equation of one-dimensional consolidation is

$$\frac{\partial^2 P_w}{\partial x^2} = -\frac{\gamma_w}{K} \frac{\partial E_y}{\partial t} \qquad \text{.....(5.2)}$$

where P_w = pore water pressure, and

E_y = unit strain in y-direction.

But Equation (5.2) can be rewritten as

$$\frac{\partial^2 P_w}{\partial x^2} = - \frac{\gamma_w}{K} \frac{1}{1+e} \frac{\partial e}{\partial t} \qquad \text{.....(5.3)}$$

where e is void ratio and a function of σ',

so that

$$e = e(\sigma')$$

or $$\frac{\partial e}{\partial t} = \frac{de}{d\sigma'} \times \frac{\partial \sigma'}{\partial t} \qquad \text{.....(5.3a)}$$

Therefore from Equation 5.3,

$$\frac{\partial^2 P_w}{\partial x^2} = - \frac{\gamma_w}{K} \; \frac{1}{1+e} \; \frac{\partial e}{\partial \sigma'} \times \frac{\partial \sigma'}{\partial t} \qquad \text{.....(5.3b)}$$

or

$$\frac{\partial^2 P_w}{\partial x^2} = - \frac{\gamma_w}{K} \; (m_v) \; \frac{\partial \sigma'}{\partial t} \qquad \text{.....(5.3c)}$$

where m_v = coefficient of volume compressibility.

Hence,

$$C_v \frac{\partial^2 P_w}{\partial x^2} = - \frac{\partial \sigma'}{\partial t} \qquad \text{.....(5.4)}$$

Now, $\sigma' = \sigma - P_w$

or $$\frac{\partial \sigma'}{\partial t} = \frac{\partial \sigma}{\partial t} - \frac{\partial P_w}{\partial t} \qquad \text{.....(5.4a)}$$

Therefore, from Equation 5.4,

$$C_v \frac{\partial^2 P_w}{\partial x^2} = \frac{\partial P_w}{\partial t} - \frac{\partial \sigma}{\partial t} \qquad \text{.....(5.4b)}$$

But from Equation 5.1,

$$\frac{\partial \sigma}{\partial t} = \gamma_t \; \frac{dh}{dt} \qquad \text{.....(5.4c)}$$

Hence,

$$C_v \frac{\partial^2 P_w}{\partial x^2} = \frac{\partial P_w}{\partial t} - \gamma_t \frac{dh}{dt} \qquad \text{.....(5.5)}$$

Equation 5.5 is the governing differential equation in terms of P_w (the total pore water pressure). It is of advantage to rewrite Equation 5.5 in terms of excess pore pressure u where

$$u = P_w - (h-y)\,\gamma_w \qquad \text{.....(5.6)}$$

From Equations 5.5 and 5.6, the governing equation can be written as

$$C_v \frac{\partial^2 u}{\partial x^2} = \frac{\partial u}{\partial t} - (\gamma_t - \gamma_w) \frac{dh}{dt}$$

or

$$C_v \frac{\partial^2 u}{\partial x^2} = \frac{\partial u}{\partial t} - \gamma' \frac{dh}{dt} \qquad \text{.....(5.7)}$$

where $\gamma' = \gamma_t - \gamma_w$,

and $u\ (x,0) = 0$ .....(5.8)

$u\ (0,t) = -\ \gamma_w(h-y).$ .....(5.9)

5.2.3.3 Solution for a Constant Rate of Deposition

For a constant rate of deposition m, Equations 5.7 to 5.9 can be rewritten as:

$$C_v \frac{\partial^2 u}{\partial x^2} = \frac{\partial u}{\partial t} - \gamma' m \qquad .....(5.10)$$

$$u\ (x,0) = 0, \qquad .....(5.11)$$

and

$$u\ (0,t) = -\gamma_w mt\ . \qquad .....(5.12)$$

To determine distribution of excess pore pressures in the thin layer dy at a height of y from the base, requires a solution of Equation (5.10) consistent with initial and boundary conditions (5.11) and (5.12).

For a semi-infinite solid, Carslaw and Jaeger (1959) give

$$v = 4kt\ i^2\mathrm{erfc}\ \frac{x}{2\sqrt{Kt}} \qquad .....(5.13)$$

as a solution of the differential equation of heat conduction,

$$K \frac{\partial^2 v}{\partial x^2} = \frac{\partial v}{\partial t} \qquad .....(5.14)$$

with initial and boundary conditions:

$$v\ (x,0) = 0 \qquad .....(5.15)$$

$$v\ (0,t) = kt \qquad .....(5.16)$$

where k is a constant,

By taking, $v = u - \gamma' mt$ .....(5.17)

our Equations (5.10) to (5.12) can be rewritten as:

$$C_v \frac{\partial^2 v}{\partial x^2} = \frac{\partial v}{\partial t} \, , \qquad \text{.....(5.18)}$$

$$v\ (x,0) = 0 \qquad \text{.....(5.19)}$$

and

$$v\ (0,t) = -\ \gamma_t mt \qquad \text{.....(5.20)}$$

By substituting C_v for K and $(-\gamma_t mt)$ for kt in Equation (5.13) to (5.16), we can write a solution to Equation (5.18) consistent with initial and boundary conditions in (5.19) and (5.20) as:

$$v = 4\ (-\ \gamma_t mt)\ i^2\text{erfc}\ \frac{x}{2\sqrt{C_v t}} \qquad \text{.....(5.21)}$$

But from Equation (5.17),

$$v = u - \gamma' mt$$

Hence, Equation (5.21) can be rewritten as:

$$u = \gamma' mt - 4\ \gamma_t mt\ i^2\text{erfc}\ \frac{x}{2\sqrt{C_v t}} \qquad \text{.....(5.22)}$$

which is a solution to our governing differential equation (5.10) consistent with the initial and boundary conditions (5.11) and (5.12).

5.2.4 Rate of Seepage

5.2.4.1 General Formulation

The rate of seepage q_y from element dy (Figure 5.6) at the sand-slime interface can be determined from

$$q_y = K \frac{1}{\partial w} \left(\frac{\partial u}{\partial x}\right)_{x=0} . \quad (1.dy) \qquad \text{.....(5.23)}$$

But from Equation (5.22),

$$\frac{\partial u}{\partial x} = -4\gamma_t mt \left[\left(- \text{ierfc} \frac{x}{2\sqrt{C_v t}}\right) \frac{1}{2\sqrt{C_v t}} \right]$$

or

$$\frac{\partial u}{\partial x} = \frac{\gamma_t mt}{\sqrt{C_v t}} \left[2\text{ierfc} \frac{x}{2\sqrt{C_v t}} \right]$$

Hence at $x = 0$,

$$\frac{\partial u}{\partial x} = \frac{\gamma_t mt}{\sqrt{C_v t}} \left[2\text{ierfc}(0) \right] . \qquad \text{.....(5.24)}$$

But from the tabulated values in Carslaw and Jaeger (1959),

$$2\text{ierfc}\ (0) = 1.128.$$

Therefore Equation (5.24) reduces to

$$\left(\frac{\partial u}{\partial x}\right)_{x=0} = \frac{\gamma_t mt}{\sqrt{C_v t}} \ (1.128) \qquad \text{.....(5.25)}$$

Hence from Equations (5.23) and (5.25)

$$q_y = K \frac{\gamma_t mt}{\gamma_w \sqrt{C_v t}} \quad (1.128)\ dy$$

or

$$q_y = 1.128(K) \left(\frac{\gamma_t}{\gamma_w}\right) \sqrt{\frac{m}{C_v}}\ (h-y)^{\frac{1}{2}}\ dy \qquad .....(5.26)$$

By integrating Equation (5.26), the total discharge Q per lineal foot of dam can be computed from

$$Q = 1.128(K) \left(\frac{\gamma_t}{\gamma_w}\right) \sqrt{\frac{m}{C_v}} \int_o^h (h-y)^{\frac{1}{2}}\ dy$$

or $$Q = 1.128(K) \left(\frac{\gamma_t}{\gamma_w}\right) \sqrt{\frac{m}{C_v}} \left(\frac{2}{3}\ h^{3/2}\right)$$

or $$Q = (.75) \left(\frac{\gamma_t}{\gamma_w}\right) \sqrt{\frac{mh}{C_v}}\ (Kh) \qquad .....(5.27)$$

Equation 5.27 can be rewritten in terms of dimensionless parameters $\frac{Q}{Kh}$ and $\frac{mh}{C_v}$ as:

$$\frac{Q}{Kh} = .75\ \frac{\gamma_t}{\gamma_w} \sqrt{\frac{mh}{C_v}} \qquad .....(5.28)$$

5.2.4.2 Numerical Examples

To demonstrate the use of the seepage equation (5.28), two numerical examples are considered. In the first example, the rate of seepage through a typical tailings

dam computed by Equation (5.28) is compared with that obtained by conventional steady state seepage analysis. In the second example, the seepage through Bethlehem tailings dam is computed from Equation (5.28) and compared with that measured in the field.

<u>Example I</u>

For the first example, a large tailings dam presently under construction in British Columbia has been selected. The dam is being constructed by the centreline technique using cycloned sand. A schematic section of the dam showing an approximate flow net for the ultimate dam has been presented by Klohn (1972A) and is reproduced here in Figure 5.7.

It should be pointed out that physical dimensions have been added to the dam by the writer and were not included in the original sketch by the above author. The following typical values have been selected on the basis of the work previously reported in this thesis:

$m = 20$ ft/yr,

C_v(slimes) $= 5 \times 10^{-3}$ cm^2/sec $= 170$ ft^2/yr,

K(slimes) $= 5 \times 10^{-6}$ cm/sec $= 1.0 \times 10^{-5}$ ft/min,

γ_t(slimes) $= 95$ pcf,

and from Figure 5.7

$h = 200$ ft.

The rate of seepage per lineal foot of dam at the section shown in Figure 5.7 is computed from Equation (5.28) as:

$$Q = Kh\ (.75)\ \left(\frac{\gamma_t}{\gamma_w}\right) \sqrt{\frac{mh}{C_v}}$$

or $\quad Q = 1.1 \times 10^{-2}$ cu ft/min/ft.

The rate of discharge Q, however, is usually computed from the flow net construction as follows:

$$Q = \frac{n_f}{n_d} Kh$$

where n_f = number of flow channels

n_d = number of equipotential drops

From Figure (5.7)

$n_f = 5$ and,

$n_d = 7$.

Hence, $\quad Q = \frac{5}{7} \times 200 \times 10^{-5} = 1.4 \times 10^{-3}$ cu ft/min/ft.

It is noteworthy that the rate of discharge computed from Equation (5.28) is higher by almost one order of magnitude than that obtained by the conventional flow net technique.

<u>Example II</u>

Bethlehem tailings dam has been selected for the second example. The description of the dam has been included previously in Chapter IV. As of July, 1973, the maximum depth of slimes against the dam in the centre of the valley was about 180 ft. Therefore, the depth of slimes is taken ranging from zero near the ends of the dam to a maximum of 180 feet in the centre. The following typical values are selected for use in this analysis on the basis of data presented previously in this thesis:

$m = 20$ ft/yr,

γ_t(slimes) = 95 pcf,

C_v(slimes) = 1×10^{-3} cm^2/sec = 34 ft^2/yr,

K(slimes) = 2×10^{-6} cm/sec = 4×10^{-6} ft/min.

As the depth of slimes varies along the length of the dam, the rate of discharge Q is computed at typical sections with varying depth of slimes from Equation (5.28) and an average value determined as follows:

Q @ h = 45 ft = 264 K

Q @ h = 90 ft = 750 K

Q @ h = 135 ft = 1380 K,

and Q @ h = 180 ft = 2110 K.

From these figures, the average rate of discharge is estimated to be

$Q = 4.5 \times 10^{-3}$ cu ft/min/ft ,

and for an estimated length of dam of 4000 feet, the total discharge is

$$Q_t = 4.5 \times 4000 \times 10^{-3} \text{ cu ft/min}$$

of $$Q_t = 18 \text{ cu ft/min} = 135 \text{ usgpm.}$$

For comparison, the seepage flow through the above dam from steady state analysis is only 8.5 usgpm. According to the discussions held with the mill staff (Walmsley, 1973) seepage collected downstream of the dam as of July, 1973 was about 200 usgpm. Of this, about 100 usgpm was estimated to be seepage through the dam and the remainder from the ground water discharge in the area.

5.2.5 Comments on the Theory and Results

It has been shown that the seepage flow computed by the new method presented in this chapter is considerably larger than that estimated by the conventional analysis but quite comparable to that measured at a typical high tailings dam. In view of the simplifying assumptions made in this proposed model, however, the implied agreement between the measured and computed values of seepage flow in Example II might be somewhat fortuitous.

Of the many assumptions made in the formulation, those of constant C_v and constant K are particularly noteworthy from the point of view of input parameters in the final analysis. Both of these parameters are functions of

void ratio as shown in Figures 3.28 and 3.31, and will vary with change in void ratio as consolidation progresses. It would be a simple matter to express K at the sand-slime interface as a function of depth in the integration of Equation (5.26) to obtain the final Equation (5.28). The formulation of the governing equation can be made more precise by adopting a non-linear theory such as that presented by Barden and Berry (1965) and solutions can be obtained by numerical integration using finite difference techniques.

It is felt that the formulation based on the above non-linear theory, is likely to be rather unwieldly and beyond the needs of current practice. The simple formulation presented here can be used to estimate possible maximum flow through a tailings dam during construction by taking conservatively averaged values for C_v and K. Subsequent to completion of tailings disposal, the flow through a dam will decrease with time as the consolidation of slimes progresses, ultimately reaching a constant low value that can be estimated by the conventional analyses.

5.3 Seepage Flow Due to Free Water Against the Sand Dam

5.3.1 General

In the previous section, at the sand-slime interface the water level was assumed to be at the surface of the slimes. A situation may arise, particularly during a period

of high spring run-off, when a shallow depth of free water may pond directly against the sand face. This will result in an increased seepage flow through the dam. The problem has been investigated in this study; the details are presented below.

5.3.2 Analytical Approach

It has been indicated previously in this chapter that the conventional analyses for steady state seepage conditions are not applicable in estimating seepage flow through a high tailings dam. Steady state seepage analyses can, however, be used for estimating additional flow through a dam due to a depth of free water against the sand dam as follows.

The conventional analyses are used to determine the discharge rates through a tailings dam with and without a depth of free water. The difference between the two discharge rates can be considered to represent the additional flow due to the depth of free water. This quantity can then be added to the quantity of flow computed by the new method described in this chapter, to arrive at an estimate of total flow through the dam for a depth of free water against the sand dam.

5.3.3 Analyses and Results

In this study, a typical high tailings dam has

been selected as shown in Figure 5.8. The analyses have been carried out using the finite element program previously described in Chapter IV (Section 4.5.3.4).

For a water level at the surface of the slimes against the sand-slime interface (Figure 5.8A) seepage flow through the dam is computed to be 1.5×10^{-3} cu ft/min/ft. It is interesting to note that this figure shows an excellent agreement with that estimated by the flow net construction for a tailings dam with an equivalent height of seepage face in Section 5.2.4.2 (Figure 5.7). It shows, therefore, that due to the impeded drainage through the slimes, the quantity of seepage flow is not affected significantly whether the water level is at the sand-slime interface or at a short distance away.

For the case of a 5-foot depth of water above the slimes at the sand-slime interface (Figure 5.8B) the seepage flow through the dam is estimated to be 1.4×10^{-1} cu ft/min/ft; larger by almost two orders of magnitude than that computed for the previous case of no free water above the slimes. The seepage flow through the slimes at the sand-slime interface, however, is 1.6×10^{-3} cu ft/min/ft (see Figure 5.8B); only slightly larger than that from the previous case. It can therefore be said that almost all the additional flow occurs through the face of the dam above the slimes where water is in direct contact with the sand.

In view of the above it appears that the seepage flow through the sand-water interface can also be computed on the basis of the work presented by Harr (1962) in his chapter on "Seepage from Canals and Ditches". Of particular interest here is his treatment of seepage from triangular-shaped ditches based on the Russian work of Vedernikov (1934) as shown in Figure 5.9. The rate of seepage from the ditch can be computed from

$$q = K(B + AH) \qquad \text{.....(5.29)}$$

where values of B, A and H are as defined in Figure 5.9.

Additional flow q_a through a tailings embankment due to a depth H ft of free water against the sand can be computed by dividing Equation (5.29) by 2 as follows:

$$q_a = \frac{q}{2} = \frac{K}{2}(B + AH) \qquad \text{.....(5.30)}$$

The values of parameters B, A and H for the problem shown in Figure (5.8) and from Figure (5.9) are

$B = 20$ ft

$H = 5$ ft

$A = 1.8$, for $\alpha = \tan^{-1}(.5) = 26.5^{\circ}$.

Hence,

$q_a = 1.45 \times 10^{-1}$ cu ft/min/ft

which shows a remarkable agreement with 1.4×10^{-1} cu ft/min/ft computed by the finite element analysis.

5.4 Phreatic Surface Within Sand Dam

It has been indicated previously in Chapter II (Section 2.4.3.5) that an impervious seal against the upstream face of the dam is considered an essential part of the design for the following two possible reasons:

i) to minimize the seepage flow through a dam for reasons of water supply and possible pollution of downstream ground water system; and

ii) to minimize the seepage flow to maintain a low phreatic surface within the sand dam for stability considerations.

So far in this chapter, the methods have been discussed for computation of seepage flow through a tailings dam; the subject of phreatic surface in the sand dam is discussed below.

In a typical tailings embankment, the phreatic surface can be located for all practical purposes according to the "Dupuit Theory of Unconfined Flow". This theory is applicable to the class of problems where the slope of the phreatic surface is relatively flat and is based on the following assumptions (Dupuit, 1863):

i) for small inclinations of phreatic surface, the streamlines can be taken as horizontal, and

ii) the hydraulic gradient is equal to the slope of the phreatic surface and is invariant with depth.

Although the nature of these assumptions appears paradoxical, in many ground water problems, solutions based on the Dupuit assumptions compare favourably with those obtained by more rigorous methods. Moreover, Charny (1956) has shown that the discharge quantity is correctly predicted by this formulation.

Within the framework of the Dupuit assumptions, for a two-dimensional flow on a horizontal impervious boundary (Figure 5.10), Harr (1962) derives the following two equations:

$$h = \sqrt{h_1^2 - (h_1^2 - h_2^2)\frac{x}{L}} \qquad \text{.....(5.31)}$$

$$q = K\,\frac{h_1^2 - h_2^2}{2L} \qquad \text{.....(5.32)}$$

Equation 5.31 describes the location of the phreatic surface and Equation 5.32 gives the discharge rate.

Numerical analyses carried out by the writer using finite element techniques indicate that Equation 5.31 can also be reliably used to determine the location of the phreatic surface in the above problem provided

$$L \geq 4(h_1 - h_2) \qquad \text{.....(5.33)}$$

and $$h_2 > 0. \qquad \text{.....(5.33a)}$$

In the case of a typical tailings dam for which the rate of seepage q has been determined by methods previously discussed in this chapter, the upstream height h_1 can be determined as follows.

From Equation 5.32,

$$h_1^2 - h_2^2 = \frac{2qL}{K} \qquad \text{.....(5.34)}$$

Generally speaking h_2 is likely to be much smaller than h_1 and can be safely assumed to be zero. Therefore, Equation 5.34 reduces to

$$h_1 = \sqrt{\frac{2qL}{K}} \qquad \text{.....(5.35)}$$

Now for example, in the case of the tailings dam shown in Figure 5.7, q has been estimated in Section 5.2.4.2 as 1.1×10^{-2} cu ft/min. The other quantities in the right hand side of Equation 5.35 for this dam as shown in Figure 5.11 are

$L = 370$ ft

$K = 1.0 \times 10^{-2}$ ft/min.

Hence,

$$h_1 = \sqrt{800} = 28 \text{ ft.}$$

The phreatic surface downstream of the starter dam can then be estimated from Equation 5.31. The phreatic surface along the starter dam has been found by finite

element analyses to be essentially parallel to the slope. It is of interest to note that the depth of water along the starter dam in the above example is only about 2 ft.

In this section, only the flow on a horizontal impervious base has been treated according to Dupuit theory. This theory can, however, be equally adapted to the flow on a sloping impervious base as discussed in detail by Harr (1962).

5.5 Impervious Seal

It has been indicated previously that in the construction of a tailings embankment by downstream methods, an impervious seal is employed against the upstream slope to minimize seepage flow through the dam.

For the consideration of an impervious seal, the problem of flow through a tailings dam can be subdivided into three categories:

i) when slime-water slurry is ponded directly against the sand as typified by the situation at the Bethlehem tailings dam,

ii) when a shallow depth of water (less than 5 feet) is ponded against the sand above the slimes for a short duration such as during periods of high spring run-off, and

iii) when a significant depth of water (15 or

20 feet) is likely to be ponded against the sand continuously as in the case of a dam where slimes are discharged at a point remote from the dam as described by Hoare and Hill (1970).

It should be recognized at this point that the need for an impervious seal at a given tailings dam should be assessed in light of particular details of the project as the requirements may vary with the nature of the materials being used. The comments included below serve to present typical situations and apply only to the numerical examples discussed previously in this chapter.

In the class of problems covered in item (i) above, the rate of discharge through a typically high tailings dam is about 135 usgpm as discussed in Section 5.2.4.2. For this order of flow, the phreatic surface is maintained near the base of the dam as shown in Figure 5.11. Most tailings disposal systems today are designed to include a catchment area downstream of the dam where seepage water is collected and pumped back into the system. It appears, therefore, that a seepage flow of 135 usgpm is not likely to pose any problem. In view of these considerations it can perhaps be said that the cost of an impervious seal is not justified in the class of problems described above.

Theoretical analyses presented in Section 5.3 indicate that additional flow through the dam due to a 5-foot depth of water against the sand above the slimes is 1.4×10^{-1} cu ft/min/ft. For a typical length of 4000 ft of dam, this will result in an additional flow of 560 cu ft/min or about 4,200 usgpm. This is a substantial increase over the discharge rate of 135 usgpm discussed in the previous paragraph. Therefore, it appears on the basis of the above analyses that an impervious seal, in this case, may be warranted. It is felt, however, that there is likely to be sufficient amounts of fines suspended in the water against the dam, particularly if the water level rises gradually and the slimes are being discharged from along the dam. Under these circumstances, the fines will sediment to the bottom and form an impervious seal. The actual design might depend upon the experience gained through on-going observations on the project.

For the class of problems covered in item (iii) above, the discharge rate for a depth of 15 feet of water, for example, will amount to 12,500 usgpm. With no suspended fines in the water due to the remote discharge of slimes, it is felt that an impervious seal is imperative in this design.

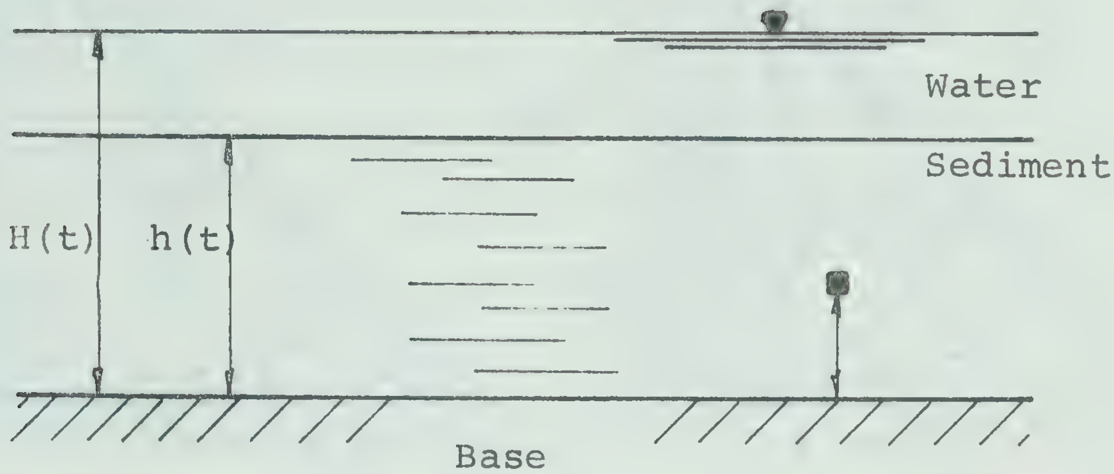


Figure 5.1 Moving Boundary Problem (After Gibson, 1958)

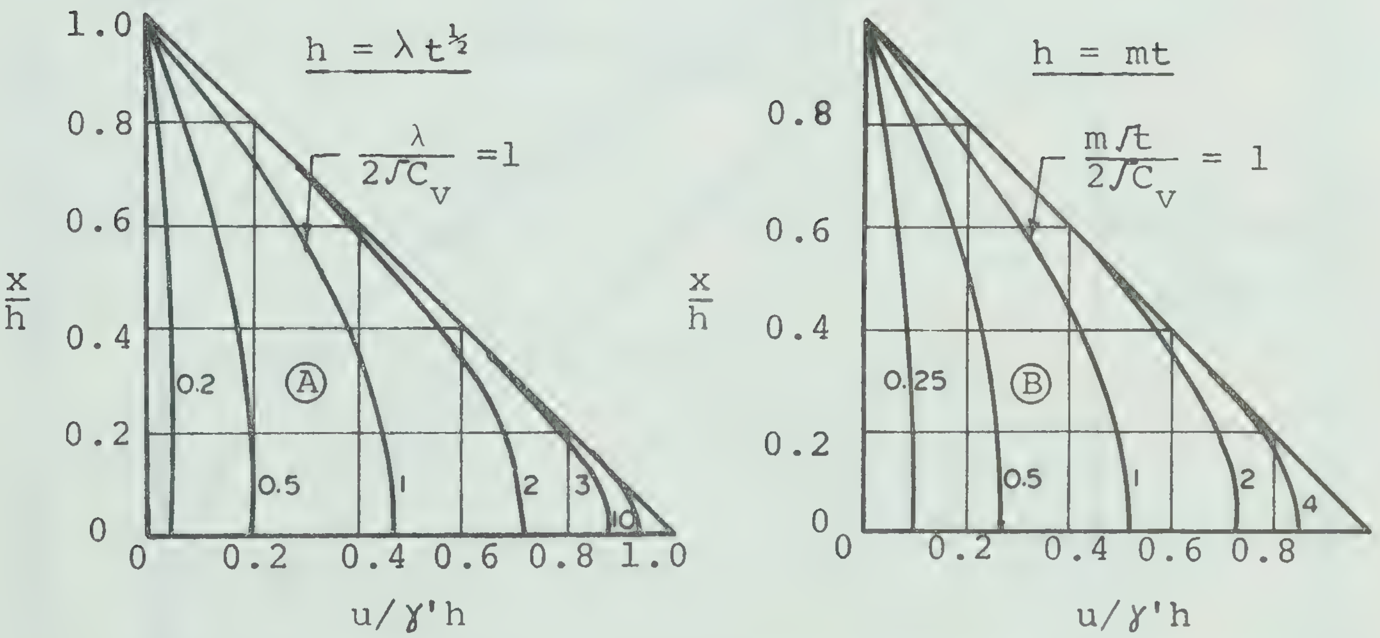


Figure 5.2 $u/\gamma'h$ Versus x/h (Impervious Base) (After Gibson, 1958)

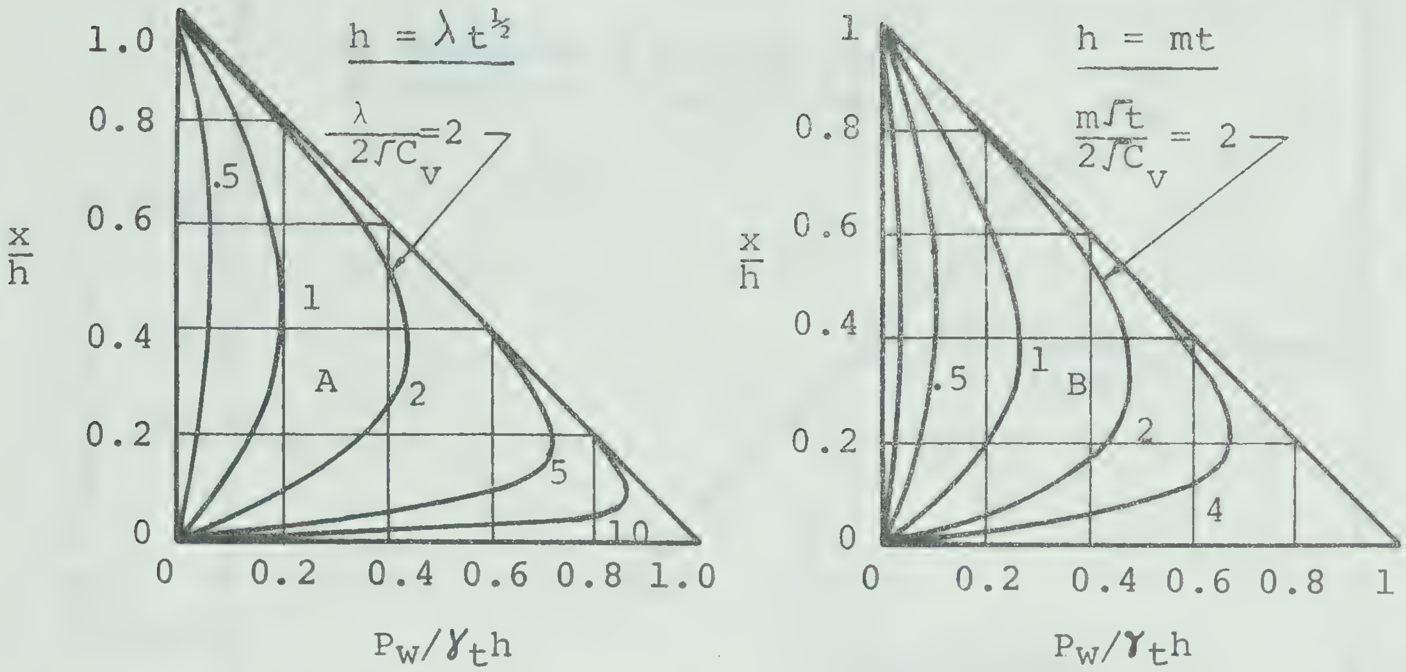


Figure 5.3 $P_w/\gamma_t h$ Versus x/h (Pervious Base)
(After Gibson, 1958)

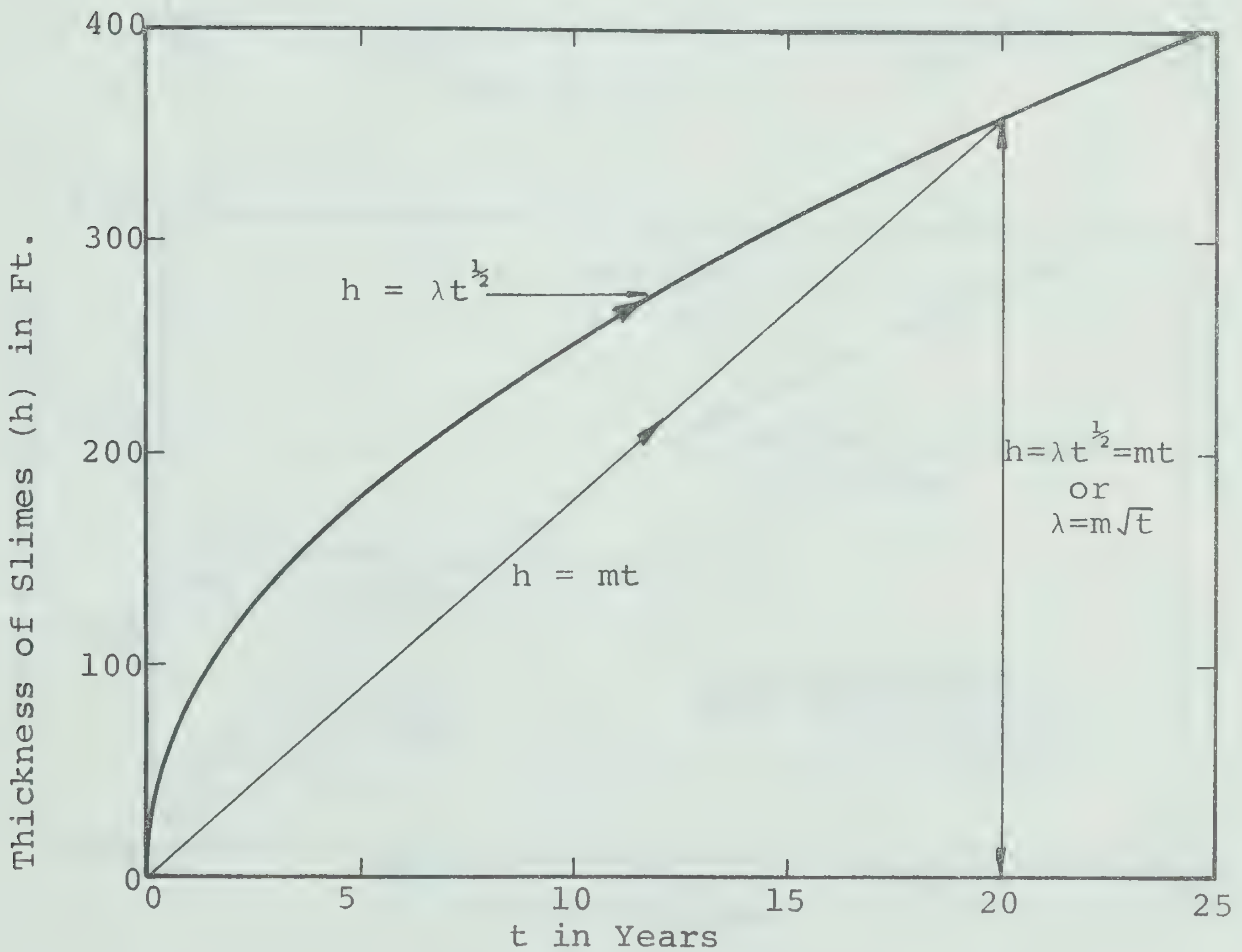
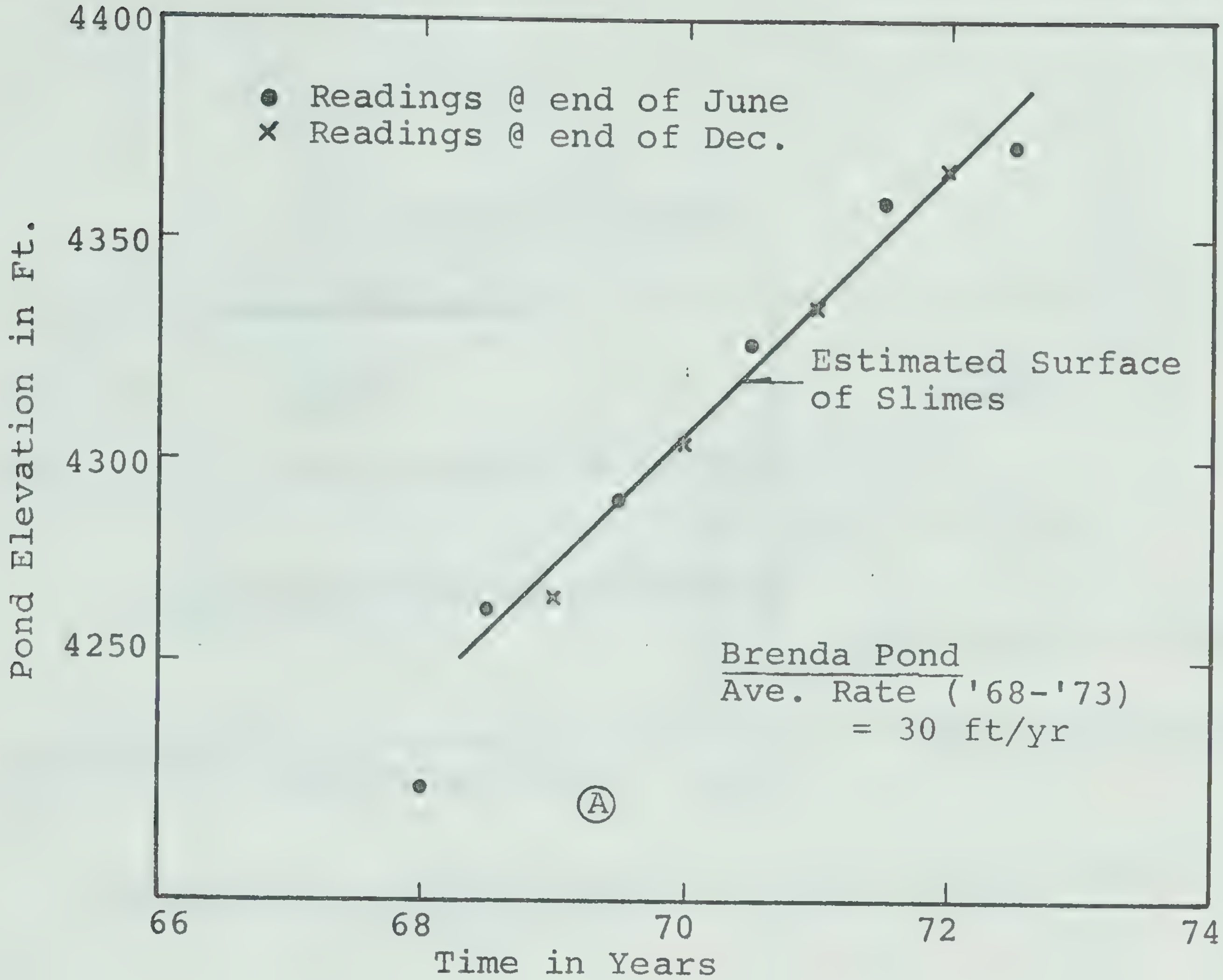


Figure 5.4 Different Rates of Deposition



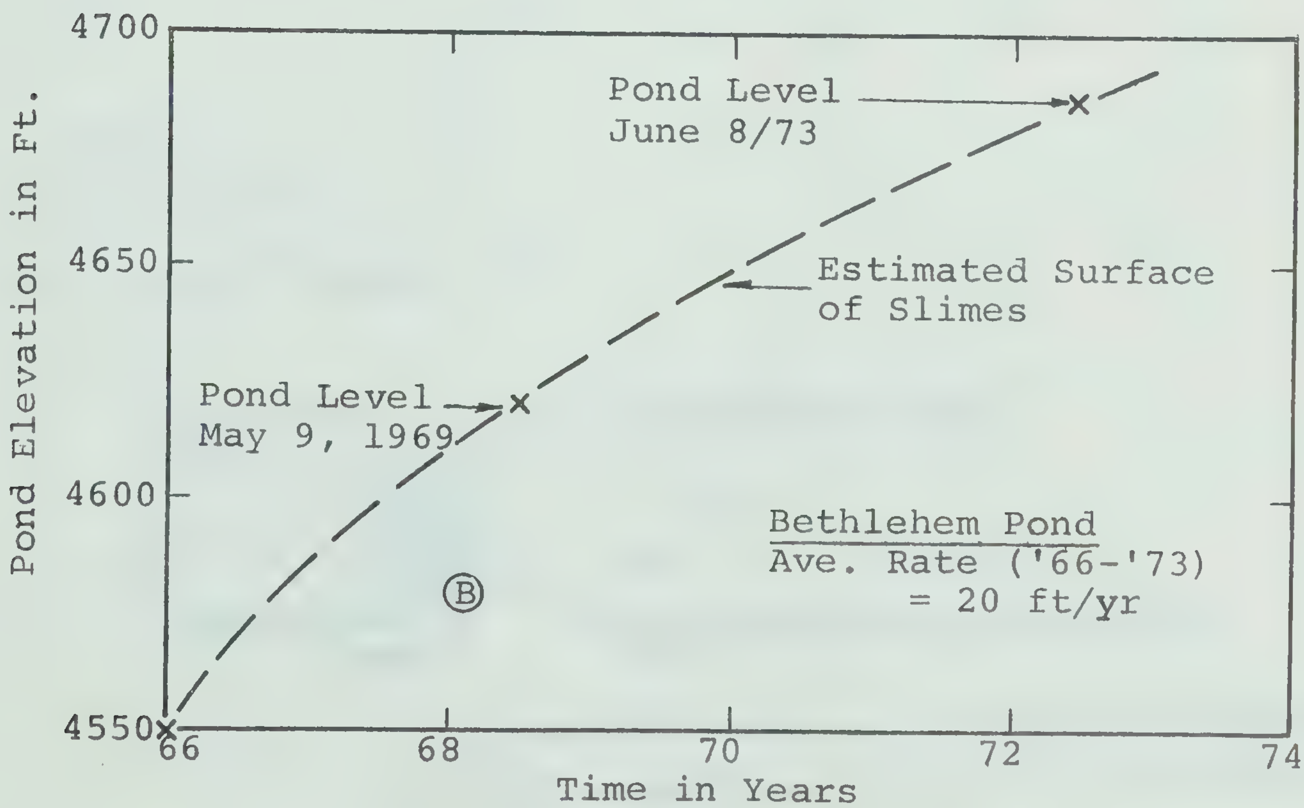


Figure 5.5 Pond Level Versus Time

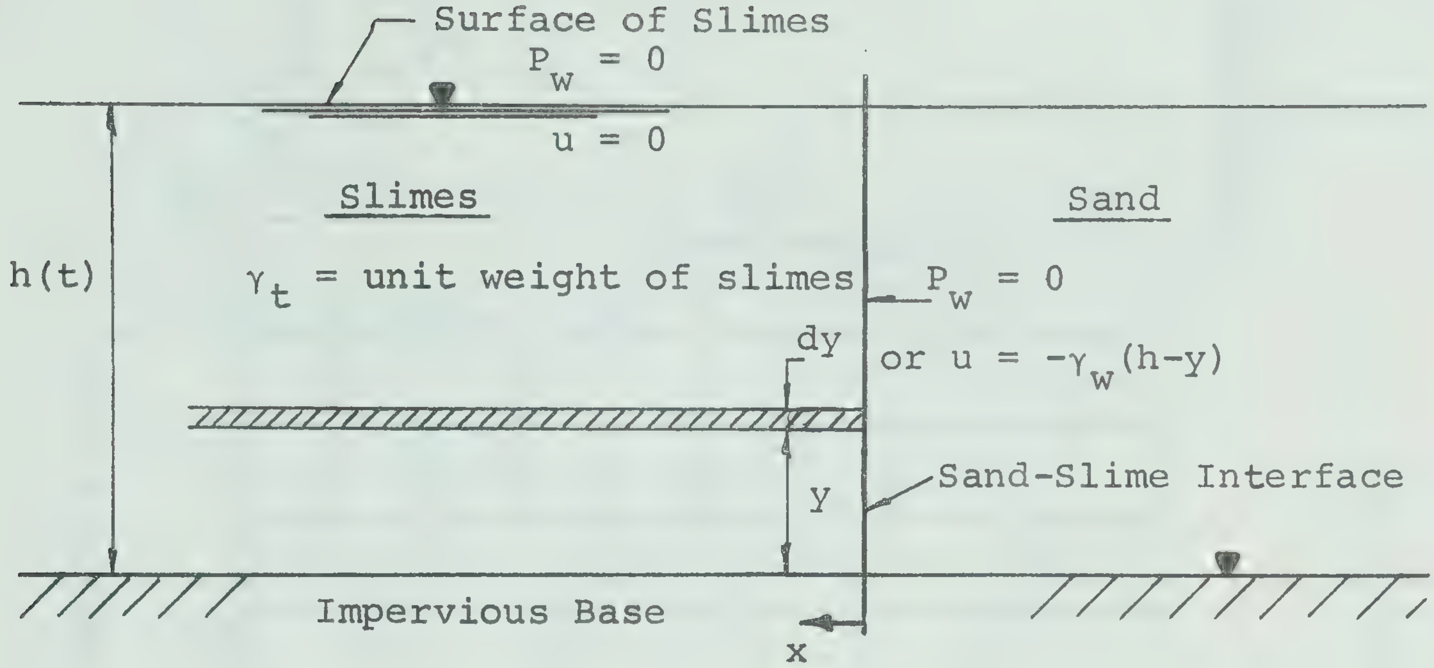
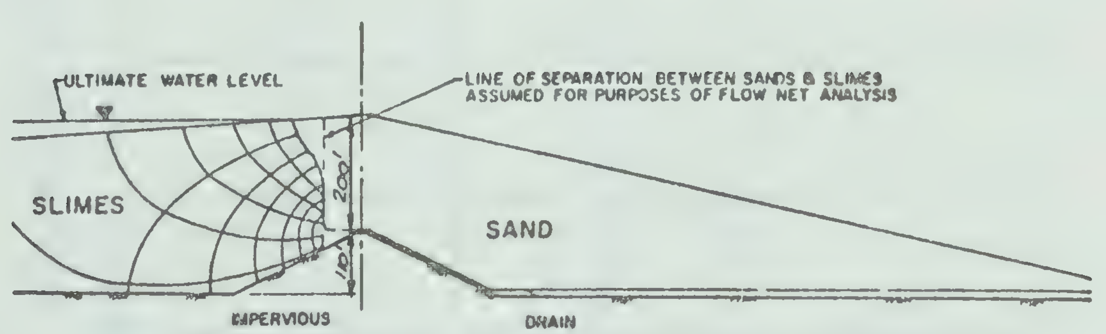


Figure 5.6 Consolidation of Slimes @ Sand-Slime Interface

slimes

$C_v = 170\ ft^2/yr$

$K = 5 \times 10^{-6}$ cm/sec



q(from flow net) = 1.4×10^{-3} cu ft/min/ft

q(from eq(5.28)) = 1.1×10^{-2} cu ft/min/ft

Figure 5.7 Approximate Flow Net for a Typical Tailings Dam (After Klohn, 1972A)

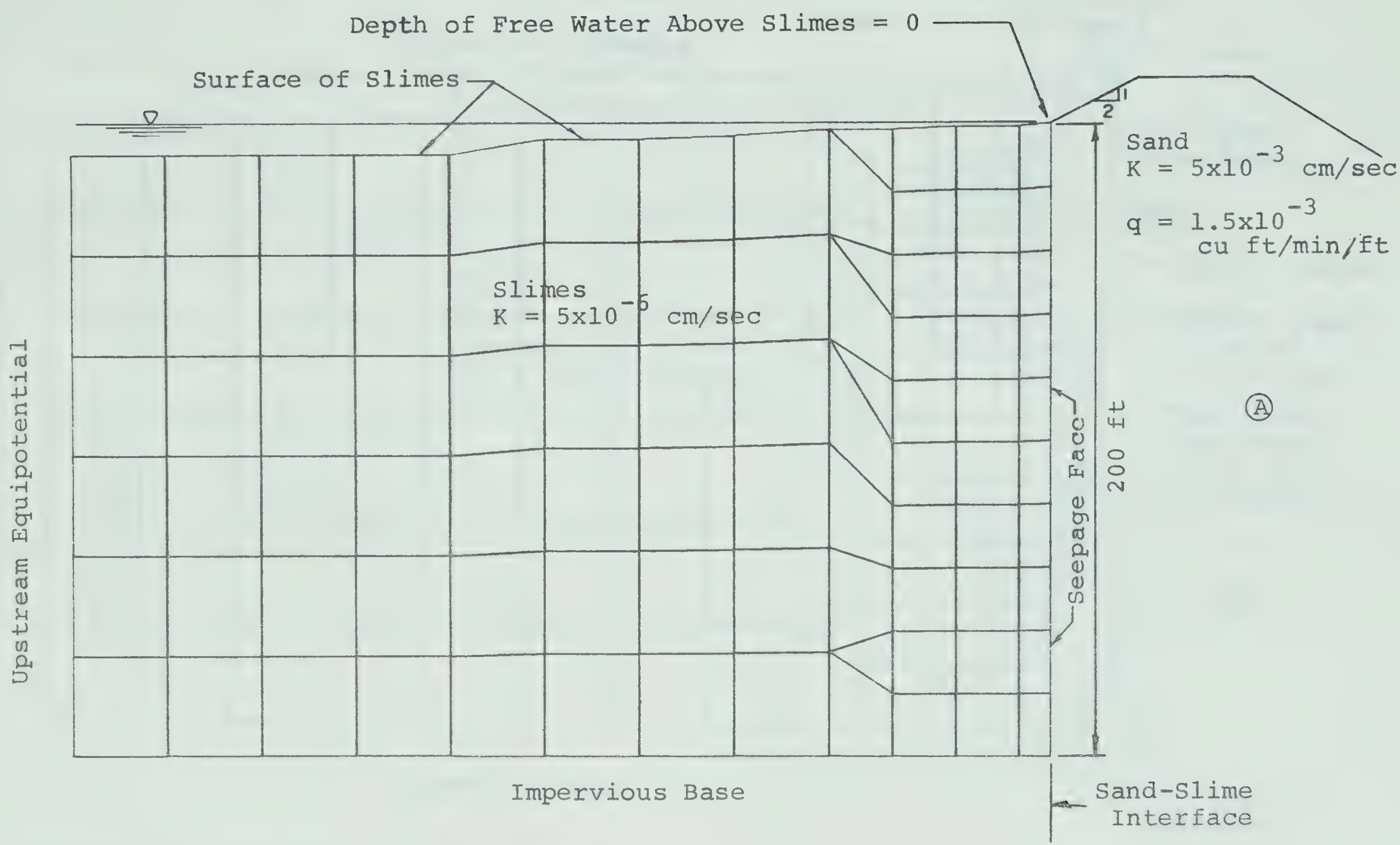


Figure 5.8 Seepage Flow Through Typical Tailings Dam (Finite Element Analysis)

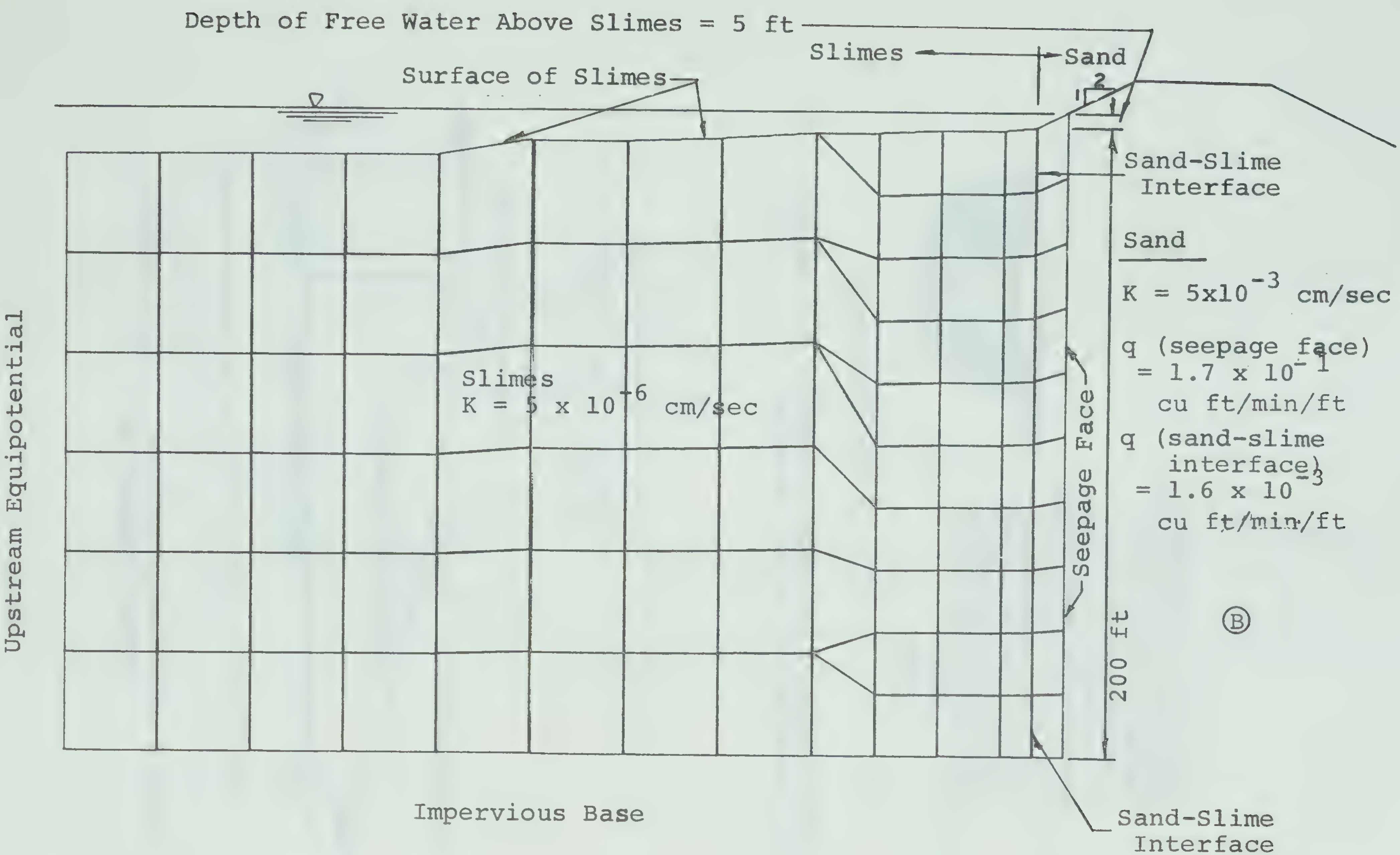
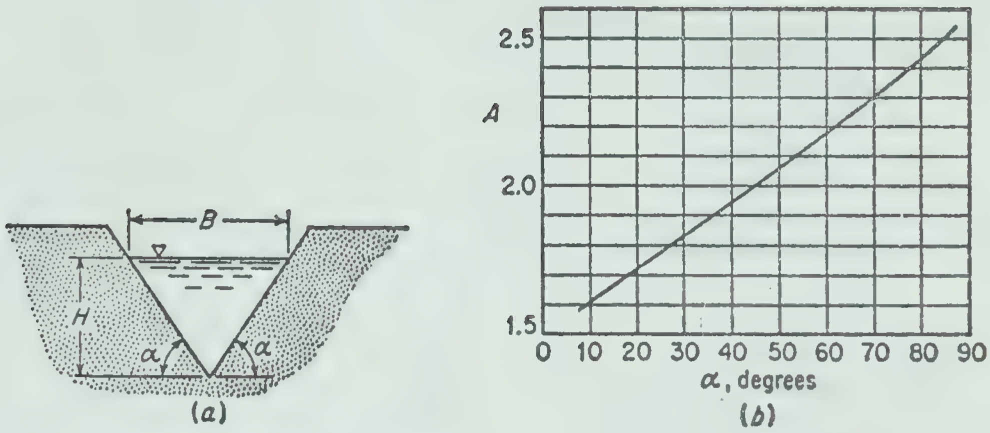


Figure 5.8 (cont'd) Seepage Flow Through Typical Tailings Dam (Finite Element Analysis)



$$q = K(B + AH)$$

Figure 5.9 Seepage From Triangular Shaped Ditches (After Verdernikov, 1934)

$$h = \sqrt{h_1^2 - (h_1^2 - h_2^2)\frac{X}{L}}$$

$$q = K\,\frac{h_1^2 - h_2^2}{2L}$$

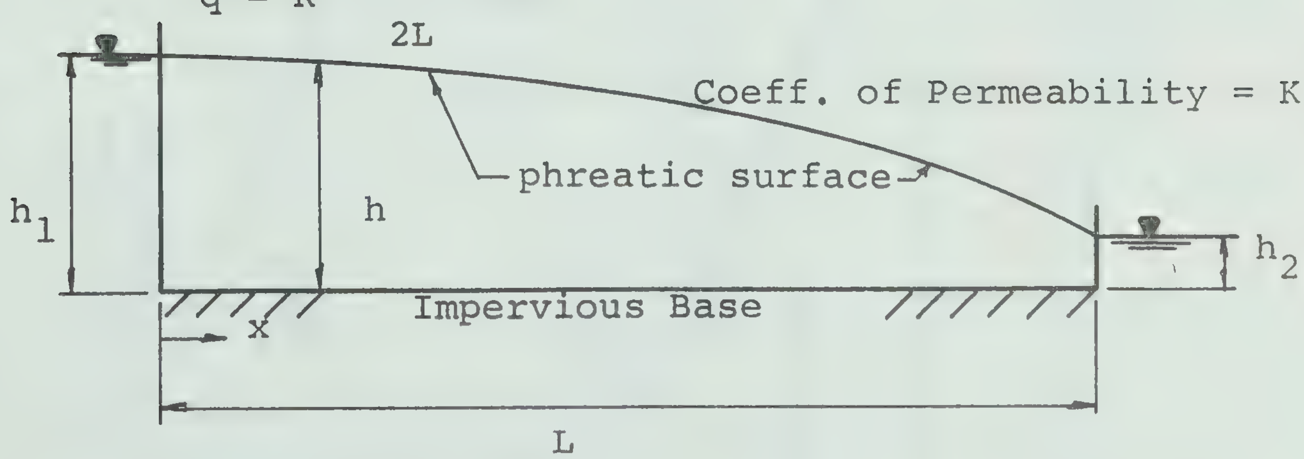


Figure 5.10 Two Dimensional Flow Problem by Dupuit Theory

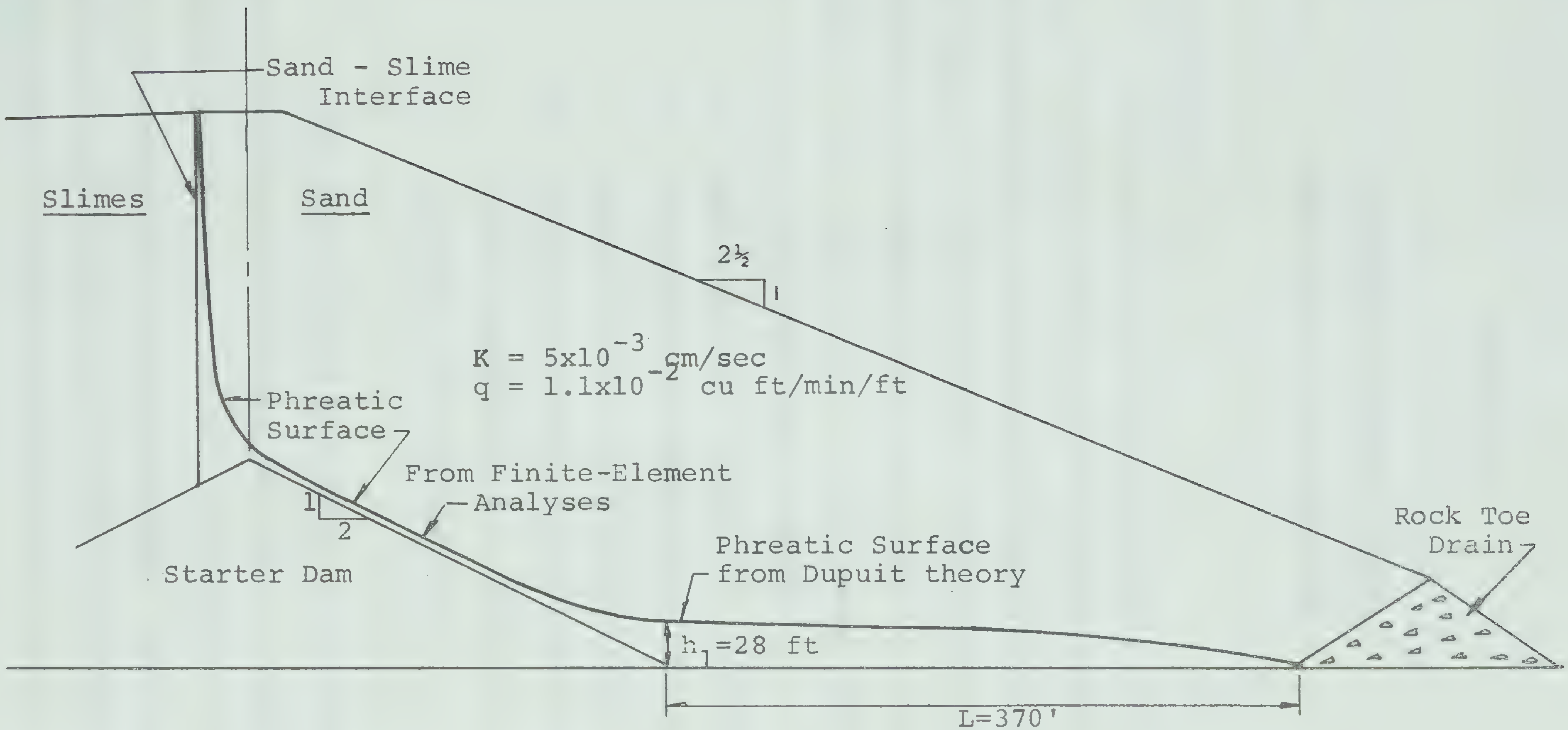


Figure 5.11 Phreatic Surface in a Typical Tailings Dam
(Reference Figure 5.7)

CHAPTER VI

EARTHQUAKE EFFECTS ON THE STABILITY OF TAILINGS DAMS

6.1 Overview

It has been indicated previously that in seismic areas, the tailings dams must also be designed to withstand shock loading due to earthquake tremors in addition to the usual stability considerations under static loading conditions. It appears that with regard to earthquake effects on the stability of tailings dams, consideration must be given to the following factors:

i) possibility of failure by liquefaction;

ii) earthquake induced shear displacements in slopes of dry cohesionless soils; and

iii) earthquake induced settlements in a deposit of dry sand or in wet sand after pore pressures are dissipated.

Failure by liquefaction is of interest in the design of tailings dams since the fine to medium grained tailings sands are particularly susceptible to liquefaction when subjected to earthquake shaking. When the tailings dams are constructed primarily of these materials, the consequences of failure by liquefaction can be very severe

[illegible]

(Dobry and Alvarez, 1967).

A review of the case histories where liquefaction has occurred indicates that liquefaction occurs only when materials are relatively loose and saturated or at least nearly saturated. Drainage and densification, therefore, would seem to be the logical protections against liquefaction. Densification of soil has been recognized as a protective measure against liquefaction since Casagrande (1936) first introduced the concept of critical void ratio. Accordingly, a series of empirical guides have been employed by various engineers and will be discussed here.

As an alternative to densification, it would appear that the drainage of soil might be used effectively to preclude liquefaction. The construction of a tailings dam can be adapted to the use of this technique. In a tailings dam, by judicious selection of materials and installation of adequate underdrains, the phreatic surface can be maintained well away from the outside slope and near the base of the dam. Under these circumstances, large portions of the sand dam are partly saturated and as such are not likely to be susceptible to liquefaction.

The slope would, however, be subjected to the possible shearing distortions caused by the earthquake induced ground accelerations.

If the sand is in a relatively loose condition in the above case of a tailings dam, the material would also be subjected to settlements induced by the vibratory loading of an earthquake.

In addition to the above considerations, liquefaction of slimes is another aspect of the subject in question. In current practice, it is generally assumed that the slimes in a tailings pond liquefy during a seismic shock; the embankment is designed accordingly to withstand increased hydrostatic pressure against the upstream slope. Whether the slimes actually liquefy or remain in a slurry form as discussed in the previous chapter is of little practical significance in this case since, in either event, the design would include full hydrostatic pressure of liquid slimes against the upstream face of the embankment. Liquefaction of slimes, therefore, is no longer an issue for further discussion in this chapter.

6.2 Criteria to Preclude Liquefaction of Sands

6.2.1 Densification

To preclude liquefaction of sand, it is generally considered sufficient to increase its density above a certain threshold value. A series of empirical guides have been established as summarized in Table 6.1. On the basis of the experimental work of Castro (1969) and many years of experience, Casagrande (A.) considers that ordinary sands

with a relative density of more than 50% would be safe against liquefaction (Green and Ferguson, 1970). In view of the analytical, laboratory and field data, Seed and Idriss (1971) present their findings on the subject in terms of ranges of values for relative densities dependent upon the maximum anticipated ground accelerations. Their results for the sites consisting of sand with a water table about 5 feet below the ground surface are summarized in Table 6.2.

With regard to the criteria cited above for elimination of liquefaction, the following points are noteworthy:

i) Some of the criteria, for example that which is attributed to Casagrande, refer to failures by liquefaction associated with large scale shear displacement such as flow failures in the cases of embankment instability. The criteria presented by Seed and Idriss, on the other hand, refer to the onset of liquefaction in the case of level ground where shear displacements would be virtually nonexistent; and therefore might tend to overestimate the densities required to avoid liquefaction if applied to the case of embankment instability. See Youd (1973) for further clarification of the definitions for various classes of liquefaction.

ii) The above criteria except those of Seed and Idriss (1971), and D'Appolonia (1970) are all inclusive in terms of earthquake intensities. For example, if Casagrande's criterion of 50% relative density is valid to avoid liquefaction under a severe earthquake it would appear to be conservative for sites which are likely to be subjected to only moderate intensity of ground shaking.

iii) The values of relative densities cited above in Tables 6.1 and 6.2 cannot be compared quantitatively, since it is more than likely that these investigators had used different test procedures to determine the limiting densities. Therefore it is entirely possible that there might be inherent differences in their computed relative densities for a given test density. For example, Castro determined his maximum density by

> "...Hammering forcefully the sides of the mold and also over a plate on top of every one of three layers."

While Seed and Idriss follow the same procedure as that followed by Lee and Seed (1967), which is

> "...By vibrating saturated sand in a mold until it reached constant volume."

Neither method is according to the ASTM procedure for determining maximum density (ASTM D-2049-69)

although the Seed and Idriss technique could be construed as being the closest. Minimum density determinations, by both of these investigators have been essentially according to the ASTM procedure.

iv) In applying these criteria to a practical problem, it should be remembered that liquefaction could occur in a buried loose layer and in turn cause instability in the overlying dense materials (Ambraseys and Sarma, 1969). In the case of tailings dams to be constructed in seismic areas, therefore, inclusions of loose materials of any extent should be avoided at all costs.

In view of the criteria noted above and the pertinent data for reported cases of liquefaction and non-liquefaction (Seed and Idriss, 1971), it appears that <u>50 to 60% relative density</u> should be sufficient to preclude liquefaction in the case of the angular tailings sands where design accelerations are not likely to exceed 0.1g. In areas of high seismicity (acceleration greater than 0.1g), however, higher densities may be required; each case must be assessed on its own merits.

6.2.2 Confining Pressure

It has been demonstrated that the higher the confining pressure, the greater the number of cycles of shear stress of a given magnitude for inducement of liquefaction (Seed and Lee, 1966). This trend is also valid for density. Accordingly, it appears that there are two basic approaches in developing criteria to preclude liquefaction:

i) to increase density, and

ii) to increase confining pressure.

For most cases, increasing soil density is more practical. Nevertheless D'Appolonia (1970) indicates that there are a number of cases where surcharging the surface and increasing the contact bearing pressure beneath the foundations will increase the confining pressure sufficiently to prevent liquefaction. Burke (1973B) indicates that such a technique has been used to reduce liquefaction potential in the case of nuclear power plants.

On the other hand, Seed and Peacock (1971) summarize the work of many investigators to indicate that for a particular sand at a known relative density, the liquefaction potential in a given number of cycles of shear stress is a function of stress ratio (τ/σ_a) independent of the confining pressure where

τ = applied dynamic shear stress,

and σ_a = initial confining pressure.

Consider, for example, an element of soil at a depth of h ft below the ground surface. If the water table is assumed at the ground surface, the effective overburden stress is $\gamma' h$. Therefore the average initial stress is

$$\sigma_a = \gamma' h \left(\frac{1 + 2\,K_o}{3}\right) \qquad \text{.....(6.1)}$$

where $\gamma' = \gamma_t - \gamma_w$,

and K_o = coefficient of earth pressure at rest.

It is generally accepted that the shear stresses developed at any point in a soil deposit during an earthquake are due primarily to the upward propagation of shear waves in the deposit. Therefore the shear stress due to a ground acceleration a can be computed by

$$\tau = \gamma' h \; a/g \qquad \text{.....(6.2)}$$

where g = acceleration of gravity.

Hence the stress ratio from Equations 6.1 and 6.2 is

$$\frac{\tau}{\sigma_a} = \frac{a}{g} \left(\frac{3}{1 + 2K_o}\right) \qquad \text{.....(6.3)}$$

Now if a uniformly distributed surcharge load P is applied at the surface of the deposit, the effective overburden stress at the point in question will be $\gamma' h + P$ after excess pore pressures have dissipated. The average initial

confining stress will be

$$\sigma_a = (\gamma' h + P) \left(\frac{1 + 2K_o}{3}\right) \qquad .....(6.4)$$

and shear stress

$$\tau = (\gamma' h + P)\ a/g \qquad .....(6.5)$$

Therefore the stress ratio will again be

$$\frac{\tau}{\sigma_a} = \frac{a}{g} \left(\frac{3}{1 + 2K_o}\right) \qquad .....(6.6)$$

Since the stress ratio does not change on the application of surcharge load, it appears from the work of Seed and Peacock (1971), that the liquefaction potential at the point in question does not change.

Any further discussion on the subject is beyond the scope of this thesis; it is felt, however, that in view of the above conflicting evidence, the issue of increasing confining pressure to reduce liquefaction potential has not been resolved to the point where it can be reliably used in design.

6.3 Earthquake Effects on Dry Sand Slopes

6.3.1 General

Since unsaturated sands do not liquefy, it has been indicated that drainage of water from the hydraulically placed tailings sands can be used to eliminate liquefaction.

If the sand is of a relatively high permeability and the seepage through the dam is kept to a minimum, the phreatic surface can be maintained near the base of the dam as discussed in the previous chapter. The large percentage of sand in the embankment is therefore unsaturated and at relatively low water contents (measured water contents at the Bethlehem and Brenda dams generally ranged from about 5 to 15% except where perched water tables were encountered). The slopes comprised of unsaturated sands, however, will be subjected to earthquake induced deformations which are discussed here.

It should be noted that the zones of saturation in a dam, however, may be subject to liquefaction and should be treated accordingly as discussed previously in this chapter.

6.3.2 Stability of Dry Sand Slopes

6.3.2.1 Review of Previous Studies

Strictly speaking we are interested in the behaviour of slopes of unsaturated tailings sands under the earthquake loading conditions. However, since the tailings sands are sufficiently clean and the capillary pressures negligible, the unsaturated tailings sand in this case may be treated as a dry cohesionless material. Analysis of the stability of slopes in such materials under the influence of ground acceleration involves two steps:

i) the determination of yield acceleration, or the acceleration at which sliding will begin to occur, and

ii) the determination of deformations that will result if the imposed acceleration exceeds the yield value.

Using the sliding block analogy for cohesionless materials, Seed and Goodman (1964) show that the yield acceleration a_y can be expressed by the equation

$$a_y = \frac{\text{Sin } (\phi' - \alpha)}{\text{Cos } (\phi' - \alpha - \theta)} \, g \qquad \text{.....(6.7)}$$

where ϕ' = angle of shearing resistance of sand,

α = inclination of slope, and

θ = inclination of ground acceleration toward the slope below the horizontal.

For a horizontal ground acceleration this expression reduces to

$$a_y = \tan (\phi' - \alpha) \, g. \qquad \text{.....(6.8)}$$

For a slope subjected to ground acceleration, yielding will begin when the acceleration exceeds the value a_y. The extent of the ensuing displacements will depend on the nature of the ground motions and the stress

deformation characteristics of the slope material. Newmark (1965) suggests that these displacements may be computed by a procedure analogous to that used for analyzing the deformations of a sliding block on an inclined plane. Further analyses and model studies by Goodman and Seed (1966) confirm the validity of Newmark's approach. It should be noted however that all model studies reported by Seed and Goodman (1964), and Goodman and Seed (1966) were performed on dense materials. Observations of interest here from these studies can be summarized as follows:

i) if the slope is initially steeper than the angle of friction of the soil in a loose condition, then failure usually develops in the form of a "sand run" -- an unlikely event in the case of a well designed tailings dam as stability considerations would generally dictate flatter slopes than the angle of friction of the loose sand;

ii) in a well compacted slope, experimental evidence indicates that the failure occurs by mass sliding of a thin surface zone; and

iii) if the initial inclination of the slope is less than the angle of friction of the soil in a loose condition, movement occurs only during the application of inertia forces and no further displacement occurs when the ground motion has ceased.

6.3.2.2 Comments Relevant to Tailings Dams

It has been assumed in the above studies that the materials are sufficiently dense so that the displacements occur only due to sliding in a manner similar to that of a block sliding on an inclined plane. It is felt that the above approach is equally applicable to the slopes containing sands at lower densities, provided consideration is also given to the settlements which might occur due to the vibratory loading of an earthquake. This aspect of earthquake effects on dry sand is discussed later in this chapter.

It has been indicated that the ground acceleration must exceed the yield value before any yielding or sliding can occur. In the case of a typical tailings dam, consider a downstream slope of about 22^{o} (2.5 to 1.0, horizontal to vertical) and a ϕ' of 35^{o}. From Equation (6.8), the yield acceleration in a horizontal direction is

$$a_y = \tan\,(35^{o} - 22^{o})\ g = .23g$$

Therefore, the displacements due to sliding are likely to occur only in areas of extremely high seismicity ($a > .23g$). According to the analyses presented by Newmark (1965), for an extremely severe earthquake ($a = .5g$) of a long duration, the maximum displacement due to sliding for the above slope will be less than about 1 ft. On the basis

of the above observations it can be concluded that generally the earthquake induced displacements of the type in question would not be an issue in the design of tailings dams.

6.3.3 Earthquake Induced Settlements

In addition to the sliding displacements discussed in the previous section, significant settlements can also occur in dry or moist cohesionless soils as a result of the compaction induced by the earthquake shaking. Seed and Silver (1972) outline a procedure for estimating these settlements. The procedure takes into account the initial density of the deposit and the initial stress conditions as well as the intensity and duration of an earthquake. The proposed procedure has provided reasonable values of settlement for sand layers in shaking table tests.

A typical example of the use of the method is provided for a 50-ft thick sand deposit having a relative density of 45% subjected to a maximum base acceleration of about 0.35g as recorded on a site in the San Fernando earthquake of 1971. The computed settlement of 2½ inches is generally consistent with the settlements observed in this earthquake.

Different sand will clearly have different compaction characteristics. Each case must therefore be assessed separately. For a typical tailings dam, however,

where relative density of hydraulically placed sand is likely to be about 45%, the above computed settlements are extrapolated to obtain a rough estimate of the anticipated settlements. Accordingly for a 300 ft high tailings dam, the estimated settlements under an equivalent earthquake will be about 12 inches. The settlements of this magnitude are not likely to pose any serious problem in the design of tailings dams.

6.4 Summary of Conclusions

1. On the basis of the information presented in this chapter it has been concluded that a relative density of 50 - 60% should be sufficient to preclude liquefaction in the case of tailings sand where design accelerations are not likely to exceed 0.1g. In areas of high seismicity, higher densities may be required. Inclusions of loose materials of any extent should be avoided in the case of tailings dams to be constructed in seismic areas.

2. It has been indicated that as an alternative to densification, drainage of tailings sand may be used effectively to reduce liquefaction potential.

3. It appears that earthquake induced instability in dry cohesionless slopes only causes shallow displacements of insignificant magnitude.

4. Earthquake induced settlements are likely to be of insignificant magnitude in the case of tailings dams.

TABLE 6.1 Summary of Empirical Criteria To Preclude Liquefaction of Sand

Reference	Density Requirements to Preclude Liquefaction
Sherard, et al (1963) (p 433)	Sand with a relative density of 50% or more probably cannot liquefy regardless of the gradation.
Casagrande L. (1971)	As far as compaction of mine tailings dams is concerned, a relative density of 60% is sufficient.
D'Appolonia (1970)	Liquefaction might occur for soils having a relative density less than 50% during ground motions with accelerations in excess of approximately 0.1g. For relative densities greater than 75%, liquefaction for most earthquake loadings is unlikely.
Department of Energy, Mines and Resources (1972)	Minimum relative density of 60%.

TABLE 6.2 Ranges of Relative Density For Various Conditions of Liquefaction

Maximum Ground Surface acceleration	Liquefaction Very Likely	Liquefaction Potential Depends on Soil Type and Earthquake Magnitude	Liquefaction very unlikely
0.10 g	Dd <33%	33% < Dd < 54%	Dd >54%
0.15 g	Dd <48%	48% < Dd < 73%	Dd >73%
0.20 g	Dd <60%	60% < Dd < 85%	Dd >85%
0.25 g	Dd <70%	70% < Dd < 92%	Dd >92%

(After Seed and Idriss 1971)

CHAPTER VII

SYNTHESIS OF DESIGN CRITERIA AND MATERIALS HANDLING

7.1 Introduction

Design requirements, construction methods and procedures of materials handling to be followed at a site are three closely related aspects of constructing a tailings dam. It is futile to discuss one without a reference to the other two. For presentation purposes in this chapter, a discussion on the design criteria and materials handling requirements will be divided into three sections under the following headings:

i) upstream method of construction,

ii) downstream methods of construction, and

iii) suggested new techniques.

The class of problems to be discussed in items (i) and (ii) will relate to the conventional methods of construction as indicated by the headings. The class of problems in item (iii) will relate to the new techniques to be presented on the basis of the observations made during this research program.

7.2 Upstream Method of Construction

7.2.1 General

As previously discussed in Chapter II, the major advantage of the upstream method is its simplicity and the resulting low cost of construction. Most of the existing tailings dams in Canada appear to have been constructed by this method. Many failures, however, have been ascribed to this type of construction.

It appears that most of the previous investigators have concentrated on finding reasons why tailings embankments constructed by this method are subject to failure. It is proposed in this section to delineate operating conditions under which this technique might be used to construct safe tailings dams. Previous work on this subject has been reported by Blight (1969), Kealy and Soderberg (1969), and Kealy (1973). The work of these investigators will be discussed and incorporated where applicable.

7.2.2 Relevant Analyses

A general discussion of the design criteria for the construction of tailings dams has been included previously in Chapter II. The effect of the phreatic surface on the stability of downstream slope of a tailings dam has been demonstrated by infinite slope analysis. The cases discussed are those of a slope of cohesionless soil. In this analysis, the factor of safety for a slope with parallel seepage flow

(i.e. when the phreatic surface coincides with the surface of the slope) can be expressed by the equation,

$$F.S. = \frac{\gamma_t - \gamma_w}{\gamma_t} \frac{\tan \phi'}{\tan \alpha} \qquad .....(7.1)$$

and for a condition of no seepage flow by,

$$F.S. = \frac{\tan \phi'}{\tan \alpha} \qquad .....(7.2)$$

Kealy and Soderberg (1969) have performed stability analyses using the method of slices (Bishop, 1955) on the downstream slope of a typical tailings dam constructed by the upstream technique. The results for the cases of high and low phreatic surfaces are as illustrated in Figure 7.1. The factors of safety for the critical slip surfaces are shown for the two cases, i.e., 1.14 for the case of high phreatic surface and 2.68 for the case of low phreatic surface. It is of interest to note that the factors of safety for the above two cases from infinite slope analysis (Equations 7.1 and 7.2) for equivalent soil parameters but with (c'=0) are 1.17 and 2.10 for the high and low phreatic surfaces respectively. The discrepancy in the factors of safety for the low phreatic surface case in the two analyses is obviously due to the c' = 3.5 psi being used in the Bishop analysis and not in the other. Otherwise the agreement between the results from the two analyses is excellent.

Rather conflicting results have been presented by Blight (1969). He treats the slope as a cutting in a normally consolidated clay with the water table at the surface and a linear increase in shear strength with depth. The solution is obtained for a deposit increasing in thickness at a constant rate m by coupling the works of Gibson (1958) and Gibson and Morgenstern (1962). Sets of curves, as shown in Figure 7.2, are generated for the following input parameters:

C_v = 3000 ft^2/yr,

ϕ' = 35^o

c' = 0,

γ_t = 120 pcf, and

F.S. = 1.5.

An examination of these curves indicates that they could not possibly be correct. For instance, for a factor of safety of 1.5, the curves in Figure 7.2A give a safe slope height of 100 ft at a slope angle of 35^o for a rate of deposition of 5 ft/yr. From an infinite slope analysis for the same dam with the water table at the surface, the factor of safety is only 0.48 or alternatively, the design slope angle at a factor of safety of 1.5 is only 12.5^o. It is obvious from this example that Blight's work is in serious error. It is felt that his assumption to treat the tailings slope as a case of undrained stability is at the root of this error since a tailings slope is in fact a case of long term

(drained) stability. To put it another way, his assumption to treat a normally consolidated fill slope as a case of undrained stability is incorrect since such a slope would in actual fact be subjected to steady state seepage.

7.2.3 Existing Tailings Dams and Stability Considerations

It appears that most of the existing tailings dams have been constructed with little or no attention being paid to even the most basic principles applicable to the design of earth-fill dams. The following three items are noteworthy:

i) the downstream slopes of many existing tailings dams are too steep to provide an adequate factor of safety against a shear failure;

ii) many of the existing dams have a high phreatic surface and develop seepage through the downstream slope; and

iii) inadequate attention has been paid to the stability of foundation soils.

It has been reported that the majority of hydraulic tailings dams have an average downstream slope of 36 degrees (Hoare, 1972). The downstream slope of 19 dams at 8 different mining properties have been observed by the above author to range from 35 to 37 degrees. In arid and semi-arid regions, tailings dams with even steeper slopes have been constructed. Casagrande and MacIver (1971) describe one such dam which

has been built to a height of 300 ft at a downstream slope of about 60 degrees. Stability analyses such as those presented in the previous section would indicate that these slopes are far too steep to have an adequate factor of safety. In fact, it appears that these dams must loom dangerously close to a factor of safety of unity and would be subject to failure at the slightest provocation, such as an increase in pore pressures or softening of the exterior slope by a heavy rainfall and surface erosion or minor seismic vibrations. Perhaps this explains why so many of the failures in tailings dams have been preceded by heavy rain falls.

With regard to item (ii) above, it has been shown previously in Chapter II that the location of the phreatic surface has a marked influence on the stability of the downstream slope of a tailings dam. Accordingly, in a proper engineering design, the phreatic surface would be maintained well within the embankment and seepage water would only be allowed to exit the downstream slope through an adequately designed underdrain at the toe of the dam. Kealy and Soderberg (1969) report, however, that one of the problems observed during their visits to many mines in the United States is that the initial starter dyke is frequently constructed of impervious materials. Since the starter dyke generally forms the downstream toe of the final dam in a typical upstream construction, this results in seepage through the slope above the starter dyke and also possible

softening of the materials in the starter dyke. Hence instability develops at the toe of the dam. Out of the 19 mining operators who answered "yes" to the question "Have you had stability problems?" posed in a questionnaire by the Sub-committee on Stability of Waste Embankments (see Chapter II), nine indicated that their problem was related to seepage through the dam while most of the others indicated foundation failures.

It is of interest to note that almost 50% of the above cases of instability are related to foundation failures. It is easy to understand, however, once it is realized that 55 of the 66 mining operators who participated in the above questionnaire indicate that no subsoil investigation had been performed at the site prior to the construction of a dam.

On the basis of the above presentation, it can be concluded that the shortcomings in the tailings dams as listed in items (i) to (iii) above can be eliminated by following the basic principles of earth dam engineering. In seismic areas, however, tailings dams must also be designed to prevent liquefaction through densification or drainage as discussed in the previous chapter.

7.2.4 Materials Handling

In addition to the conditions discussed above, the other major criticism of the upstream technique is related to the possible inclusion of soft slimes within the retaining embankment. This type of problem does not lend itself readily to an adequate analysis. It appears therefore that the only practical way to solve this problem is through a proper procedure of materials handling. It has been indicated in Chapter II that in a stagnant pool of water, the tailings material sediments as a homogeneous slurry with a consistency of very soft silt. It is recognized that this phenomenon is responsible for the inclusions of soft slimes within the embankment. This happens either during the spring runoff when the length of beach is reduced or due to back-ponding during spigotting operations as described previously in Chapter II.

It is clear, therefore, that by maintaining a sufficient length of beach at all times and by careful spigotting, slimes can be kept out of the embankment area. It should be recognized, however, that a certain amount of experimentation might be required on a site before a suitable technique is evolved. Separation of sand by cyclones might be of value in this regard.

7.2.5 Concluding Remarks

It has been indicated that by following the basic principles of dam engineering and through the judicious handling of materials, safe tailings dams can be constructed by the upstream method of construction. The height to which such a dam can be constructed would however depend upon the length of beach that can be maintained. For instance, in large ponds a beach width of 200 to 300 ft should be feasible. For a beach width of 300 ft, therefore, a dam height of 120 ft is possible at a slope inclination of 2.5 to 1.0 (or slope angle of about 22°). In the case of a well drained and adequately compacted embankment, the possible height might be raised to 150 ft at a slope angle of 2.0 to 1.0 or about 26°. The actual design side slope for an embankment, however, would depend upon the angle of shearing resistance of the material and the other related factors previously discussed in this thesis. Much higher tailings dams, however, can be constructed by essentially following this method but with some modifications to the basic techniques to be discussed later in this chapter.

Construction by the upstream method is likely to be viable particularly under the following conditions:

i) when the tailings are of a relatively coarse grind (-#200 material <50%);

ii) when the dam is to be raised at a relatively slow rate (5 to 10 ft/yr); and

iii) when the dam is to be supported on a relatively pervious foundation.

Figure 7.2A illustrates a safe tailings dam being constructed by the upstream technique to meet all design requirements.

7.3 Downstream Methods of Construction

7.3.1 General

In recent years, due to the advances in technology and mining equipment, and the increasingly higher metal prices on the world market, there has been a trend towards the development of large low-grade open pit mines. Typically, each of these mines are likely to produce in excess of 200 million tons of tailings requiring some exceptionally high tailings dams.

It has been previously indicated that safe tailings dams might be possible to only limited heights by the conventional upstream construction. Therefore, for the fast rising tailings ponds of these large mines, particularly in the narrow mountain valleys, the newly evolved downstream methods are generally recommended for the construction of tailings dams. A description of these methods (downstream and centreline) has been presented in Chapter II.

Currently, tailings dams to be constructed by the

downstream methods are designed on the basis of experience available from the design and performance of conventional water storage dams. Although the basic principles of earth-dam engineering are applicable to the design of tailings dams, it should be, however, recognized that there are some fundamental differences in these two types of structures as reviewed in Chapter II. Therefore, for an optimum design, it is felt that all pertinent requirements should be evaluated in light of the specific conditions and variables involved in the construction of tailings dams. The requirements for seepage control and those relating to the earthquake resistant design have already been discussed in the previous chapters. The requirements relating to the selection and placement of materials and those of materials handling are discussed here.

7.3.2 Separation Requirements for Sands

7.3.2.1 Characteristics of Sand

As indicated previously, perhaps the most critical factor affecting the economics of constructing a tailings dam by the downstream methods is the percentage of tailings which is available as suitable construction sand for the dam. No definite criterion is available to assess the quality of sand required for the construction of tailings dams. It is indicated, however, that the sand must be free draining and it is currently believed that the amount of fines control the quality of sand. As in the case of

conventional hydraulic fills, emphasis is generally placed on getting the sand as clean as possible. Grain size distribution curves for typical hydraulic sand fills are shown in Figure 7.3. A review of the sands being used at some of the new tailings dams appears to indicate that the fines content has been restricted to less than about 12% (Table 2.2). Furthermore, in some cases, the fines content has been reduced to as little as 3%.

Although it may be desirable to have the sand free of fines from the point of view of ease of construction, the issue deserves careful consideration since the amount of fines in the sand also controls the supply of sand available for the construction of a dam. In fact, the fines content has a dual effect - an increase in the acceptable amount of fines in the sand enhances the sand supply and also reduces the amount of sand required due to the reduction in the amount of slimes to be stored. Under these circumstances, a thorough examination of the point in question is, therefore, in order.

Accordingly, a review of the possible requirements from the point of view of design and materials handling indicates that the sand must meet the following requirements:

i) the permeability of sand must be sufficiently higher than that of the slimes so that a low phreatic surface can be maintained within the

dam as discussed in Chapter V, and

ii) the sand must be of sufficiently high permeability to allow relatively rapid drainage of the hydraulically placed sand during and after placement.

In assessing the first requirement it appears that the required permeability of sand would be a function of the permeability of slimes at the sand-slime interface. Although the selection of a definite figure would be somewhat arbitrary, a ratio of 100 between the two permeabilities has been used at some dams as a criterion for quality control. On the basis of the analyses performed during this investigation, a sand-slime permeability ratio of greater than 100 or alternatively for average slimes, a sand permeability of 1.0×10^{-3} cm/sec would ensure a relatively depressed phreatic surface immediately downstream of the sand-slime interface. For instance, in the numerical example presented in Section 5.4 (Chapter V), the computed depth of water along the slope of the starter dam is only 10 ft for the sand permeability of 1.0×10^{-3} cm/sec. It can be argued, however, that the permeability of sand at depth is likely to be lower than that of the initially placed sand. For example, the permeability of sand as measured during the field testing at the Bethlehem Dam is about 3.0×10^{-3} cm/sec near the surface and as low as 0.4×10^{-3} cm/sec at a depth of 40 ft. The permeability of the sand in question might therefore be

as low as 1.0 x 10^{-4} cm/sec if a similar reduction is assumed or even lower at greater depths during the later stages of dam construction. Under these circumstances, it would appear advisable to lower the phreatic surface by including an underdrain along the downstream slope of the starter dyke or alternatively by constructing the downstream slope itself of free draining material. Therefore the phreatic surface need not as such depend entirely on the permeability of sand.

A method for locating the phreatic surface downstream of the starter dam based on the seepage flow through the dam has been presented in Chapter V. Under the average construction conditions, however, this technique is likely to give misleading results, since drainage from the hydraulically placed sand is not included in the formulation. It can be further added that the quantity of seepage from the sand could be substantial and may, therefore, control the location of the phreatic surface at least periodically. Although the situation does not lend itself to a proper analysis, for all practical purposes, the problem can be solved by incorporating an underdrainage system in the design. The phreatic surface downstream of the starter dam can, therefore, be maintained near the base of the dam by the installation of underdrains and again need not depend entirely upon the value of the sand-slime permeability ratio.

The materials handling requirement in item (ii) above can be evaluated by considering two conditions - one that during sand placement all excess water must drain vertically down without ever emerging to the surface (this is necessary to avoid possible segregation of fines) and the other that the sand must drain rapidly so that mechanical equipment can be operated on the newly placed sand within a reasonable length of time if necessary for purposes of materials handling. A discussion of these two conditions follows.

<u>Condition I</u> - Typically, sand is placed in a tailings dam by hydraulic methods. When cyclones are used to classify sand, two alternate methods of sand placement may be followed - one by direct underflow discharge from the cyclones placed on the dam and the other by conventional hydraulic fill methods by piping the sand-water mixture from the cyclones placed on the abutment in a manner similar to that followed at the Brenda dam. In the second alternative, when the sand is to be placed by the hydraulic fill methods, a large amount of water has to be added to the sand for transportation purposes. When the sand-water mixture is discharged on the dam, the sand particles immediately fall to the bottom while the finer particles remain suspended in the excess water and flow to a low area where most of the excess water is removed leaving a shallow depth of water and some fines behind. Figure 7.4 is a photograph of such

an accumulation of water on the otherwise very clean sand and Figure 7.5 is a photograph of the thin coating of fines left on the sand after the water has evaporated. If this method of sand placement is to be followed, it is quite obvious that the sand must be extremely clean to avoid large scale contamination of the sand by these thin layers of slimes which will be prime areas for perched water tables in the dam. This conclusion is in concurrence with the experience available from conventional hydraulic fills.

If sand is to be placed directly from the cyclones located on the dam, a larger percentage of fines can be accepted in the sand since it is discharged on the dam with a much lower water content. Although the sand is placed as a slurry by this method, it is not really a case of hydraulic filling in the conventional sense; the difference being that in the conventional hydraulic filling, excess water is allowed to flow at the surface of the fill resulting in possible segregation of fines carried by the water. In this case, however, the sand must be sufficiently pervious so that all excess water in the sand drains vertically downwards without ever emerging to the surface.

Consider a typical case of sand placement by this method. The water content of the cyclone underflow is likely to be about 50%. On the basis of the experiences gained during this investigation, the initial dry density of the

sand after placement can be estimated to be about 85.0 pcf with a water content of about 36% when saturated. The permeability of the sand must therefore be large enough to drain sufficient water for a change in water content from 50% to 36% during sand placement. Consider, for instance, sand placement at the rate of α ft/hr. The amount of excess water to be drained in one hour would therefore be ($0.19 \times \alpha$) cu ft/ sq ft. The rate of drainage in a downward direction would be $K \times i$ where i, the hydraulic gradient, is equal to unity. Therefore, for complete drainage of excess water in the downward direction, K must satisfy the following expression,

$$K = 0.19 \times \alpha \text{ ft/hr}$$

or

$$K = .0016 \times \alpha \text{ cm/sec} \qquad .....(7.3)$$

It has been indicated previously that a sand permeability of 1.0×10^{-3} cm/sec would be sufficient in a typical case to maintain a depressed phreatic surface in the sand dam. It is obvious from Equation 7.3 that the rate of sand placement would be restricted to about .6 ft/hr for the above value of sand permeability. On the basis of field observations, it is felt that this rate of sand placement is likely to be more than adequate for construction at most dams.

<u>Condition II</u> - In some cases mechanical equipment is required to operate on the newly placed sand for purposes of materials handling. To maintain a reasonable construction schedule,

it may be necessary in these cases for the equipment to be able to operate in an area soon after a lift of sand has been placed. Consider a case where a lift of sand has just been placed. It can be assumed that initially the sand is fully saturated to a certain depth depending on the thickness of the layer just placed. With passage of time, water drains in a downward direction and the saturation front moves below the ground surface. The sand will be fully saturated below the saturation front and only partly saturated above it. The experiences from the construction industry indicate that construction equipment can be operated on the partly saturated sand if the water table is maintained at a minimum depth of 3 to 4 feet. Therefore, it appears that sufficient time must be allowed for the saturation front to move through a distance of 3 or 4 feet before equipment can be supported on the sand. The length of time t required for the saturation front to move through this distance is obviously a function of the permeability of sand and can be computed from the following expression neglecting capillary effects,

$$t = \frac{\gamma_d \times A \times (W_o - W_f)}{62.5 \times 100 \times K} \qquad \text{.....(7.4)}$$

Where A = distance through which the saturation front moves,

W_o = initial water content in %,

W_f = final water content after drainage (%),

and

K = permeability of sand in ft/hr.

For typical values of A = 4 ft, W_o = 36%, W_f = 15% and γ_d = 85.0 pcf, Equation 7.4 is reduced to

$$t = \frac{85 \times 4 \times 21}{62.5 \times 100 \times K} = \frac{1.14}{K} \text{ hrs} \qquad \text{.....(7.5)}$$

Therefore, the time required to drain water to a depth of 4 ft would be about 10 hours for the sand permeability of 1.0×10^{-3} cm/sec. It is felt that this rate of drainage should be more than adequate to maintain a reasonable construction schedule at most dams.

7.3.2.2 Summarizing Remarks

It has been shown so far in this section that sand with the permeability value of 1.0×10^{-3} cm/sec is more than adequately pervious to satisfy all possible requirements from the view point of design and materials handling provided care is taken to avoid segregation of fines during sand placement. It should be noted that the above cited permeability value is at a relative density of about 40 to 50% which is considered typical for hydraulically placed tailings sands. Furthermore, the above cited permeability value has been somewhat arbitrarily chosen for presentation purposes. The actual value of the acceptable permeability at a given site would depend upon the specific requirements of the project, and can be evaluated in the

manner presented above. It is felt, however, that on the basis of the work presented above, the sand permeability of 1.0×10^{-3} cm/sec can be considered a reasonable criterion for quality control under average conditions encountered in the construction of tailings dams.

It has been shown in Chapter III that the permeability of a tailings sand at a void ratio of about 0.9 can be estimated from the following expression,

$$K = 100D_{10}^2 \qquad \text{.....(7.6)}$$

Where K = coefficient of permeability in cm/sec,
and D_{10} = effective size in cm.

From this equation and the permeability value of 1.0×10^{-3} cm/sec, D_{10} is estimated to be about .033 mm. On the basis of these results a practical criterion of fines content can be defined in a way that no more than 10% of the material should pass the #400 sieve (D_{10} = .037 mm). A review of the sands studied in this investigation indicates that for the above criterion of fines content, the percentage of material finer than the #200 sieve would range from 20 to 25%.

7.3.3 Density of Tailings Sands

7.3.3.1 General

It has been indicated in Chapter VI that relative

densities of 50% to 60% may be needed to eliminate liquefaction in areas of moderate seismicity (acceleration < .1g). In areas of high seismicity, even higher densities would be needed. If these densities cannot be achieved by "normal" hydraulic placement of tailings sands additional densification by mechanical equipment would obviously be required. This would naturally add to the cost of tailings disposal. Therefore, it is important to evaluate the extent of densities that may be possible to achieve by the hydraulic placement of tailings sands.

7.3.3.2 Density of Conventional Hydraulic Fills

Only hydraulic fills derived from fairly clean sands are of interest here. In his state-of-the-art paper entitled "Hydraulic Fills to Support Structural Loads", Whitman (1970) describes this type of fill to be that which is formed using borrow materials having less than 15% fines (-#200 material). Other investigators indicate that suitable hydraulic fills could be formed by using borrow materials with a somewhat higher percentage of fines. See, for example, Turnbull and Mansur (1973).

It should be recognized that the acceptable percentages of fines noted above are those present in the borrow materials to be used for hydraulic filling. It is generally assumed that through proper sluicing techniques, the actual amount of fines remaining in the sand fill can be restricted

to a more desirable figure of about less than 10%. The criterion, it appears, is to have the sand fill as clean as possible. Sand fills with less than 5% fines content are frequently cited in the literature on hydraulic fills.

With regard to densities achieved in these fills, the experiences cited in the literature are summarized in Table 7.1. These reported values of relative densities, however, should be regarded with some suspicion. Not only can sampling influence the measured in situ density, but also there has been a general lack of agreement upon methods for determining maximum and minimum densities. A review of the literature on the subject, nevertheless, indicates that with proper control of placement details, a reasonably uniform fill of moderate density can be achieved. In connection with the hydraulic fills the following additional points are noteworthy:

i) It is generally emphasized that the quality and performance of hydraulic fills are closely related to the placement techniques followed. Recommended procedures for placement of hydraulic fills are discussed by Turnbull and Mansur (1973). It is especially important to avoid ponding of water where fines might settle out to form soft pockets or layers. Furthermore, material must not be bulldozed into low areas without subsequent sluicing; otherwise zones

of low densities will result.

ii) To obtain optimum density, it is important that the true water table is well below the surface of the fill so that there is downward drainage at all times. Densities obtained in the case of hydraulic fills placed under water are generally much lower than those noted in Table 7.1.

iii) If hydraulic fills require densities greater than those noted in Table 7.1, the required densification is generally accomplished by conventional equipment.

7.3.3.3 <u>Comments Relevant to Tailings Sands</u>

In situ density results with respect to cycloned tailings sands are summarized in Table 7.2. The results indicate that there is a large variation in the values of relative density at Brenda dam where construction is carried out by the hydraulic cell method. At the other two dams in Table 7.2, the construction is by on-the-dam cycloning and the relative density varies over a relatively narrow range of values.

On the basis of the limited information included in Table 7.2, it appears that average relative densities of 45% to 55% can be achieved by on-the-dam cycloning technique. These values are comparable to those reported in the case

of hydraulic fill sands (Table 7.1). It should be noted however, that the values of relative density reported in Table 7.2 for the Bethlehem tailings sand are equivalent to only 89 to 94% of the maximum Standard Proctor Density.

7.4 Suggested New Techniques

7.4.1 Efficient Use of Cyclone Classifier

It has been indicated earlier in this chapter that the sand yield depends on the amount of fines accepted in the sand as well as the efficiency of the separating technique. The first of these two variables has been discussed in the previous section; the second is discussed here. Various methods of separating sand from tailings have been described in Chapter II. Of all the methods discussed, it has been indicated that the use of the cyclone for this purpose shows the most promise. Therefore, we will confine our discussion here to only the use of the cyclone as a sand classifier.

The cyclone has been used both as a classifier and a thickener in the milling process by the mining industry for many years and a great wealth of experience has been gained in the operation of this device in that capacity. But very little information has been published on its use as a tailings classifier. For instance, no information has been supplied on the operation of the cyclone in the "Tentative Design Guide for Waste Embankments in Canada" (The Department

of Energy, Mines and Resources, 1972).

Recently, some work has been reported on the subject by Hoare (1972). On the basis of some very costly field tests, the following conclusions have been drawn with regard to performance trends for the cyclone with a rope* type discharge:

i) An increase in the size of the apex will increase the weight recovery as long as the underflow forms a rope type discharge. Weight recovery is the weight of the solids in the underflow expressed as a percentage of the total feed.

ii) An increase in the feed pressure increases the weight recovery.

iii) An increase in the diameter of the vortex finder decreases the amount of fine material in the underflow.

iv) A decrease in the length of the vortex finder causes a decrease in the amount of fine material in the underflow.

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* There are two types of discharge commonly associated with a cyclone underflow - a vortex discharge and a rope type discharge. A vortex discharge constitutes a wild spray and a rope type discharge is a continuous stream flow with spiral motion. For purposes of tailings classification, it is necessary that the cyclone operates with a rope type discharge.

It has also been indicated that by altering the design and operating parameters, almost any desirable separation of the solid particles can be achieved by the cyclone classifier. In addition to varying these parameters, it is felt that for all practical purposes, the desired separation can be most efficiently achieved by subjecting the material to more than one stage of cycloning and/or by recycling some of the material. This technique is illustrated by examples presented below.

Example I

Grain size distribution curves are presented in Figure 7.6 for the materials associated with a cycloning operation. It can be seen that the cyclone feed (tailings) contains approximately 50% material finer than the #200 sieve. The overflow (slimes) contains 67% fines and the underflow (sand) contains 18%. The weight recovery under these circumstances is about 33%. It is easy to see that the slimes contain about 42% sand sizes* and that this percentage of sand will not be available for dam construction. In fact this material will be wasted in the slimes and will take up much needed storage space in the pond.

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* The fines are defined as the materials which pass the #200 sieve. The sand sizes are the materials which are coarser than .06 mm according to the M.I.T. grain size scale.

It is felt that by directing the overflow product through another stage of cycloning, significant amounts of additional sand can be reclaimed for the construction of the dam. The actual increase in the amount of sand would depend on the efficiency of the second cyclone. It is realistic to assume, however, that the overflow product can be adjusted to contain less than 15% sand sizes and that the underflow product will contain about 20% fines. On the basis of these assumptions, an increase in the weight recovery of 19% is realized giving a total supply of sand of 52% for the construction of the dam. It is equally important to note that the amount of slimes has been reduced to 48% from the previous 67% - a 19% decrease. It would suggest that now only 72% as much sand will be required to maintain the crest level ahead of the pond elevation as compared to that which would have been required had the second cyclone not been inserted into the system. Therefore, the net effect of the second cyclone is to increase the sand supply by almost 58% (based on the weight of the sand) and reduce the demand by 28%.

To demonstrate the effect of all these changes, consider a hypothetical case in which 50% sand recovery is needed to meet the anticipated construction requirements. It should be noted that the slimes in this case would be only 50% of the total tailings. In the above split, it would mean 50 tons of sand and 50 tons of slimes. In the original case of only one stage of cycloning the sand portion would

be 33 tons and the slimes portion, 67 tons. In this case, the amount of sand required to provide storage space for the 67 tons of slimes would not be only 50 tons but perhaps 67 tons if a proportional increase can be assumed. Therefore, there will be a total shortage of 34 tons of sand needed to meet the new construction schedule.

The effect of inserting the second cyclone into the system is to increase the sand recovery to 52 tons or an actual increase in the amount of sand of about 58%. The overall effect, however, is equivalent to doubling the sand supply - going from a 33 ton supply and 34 ton shortage to no shortage condition. This may have been an example of an extreme case, but sand shortages can be quite real at most of the dams to be constructed by the downstream methods, particularly in the critical early stages of the operation. Brenda tailings dam, the first dam to be constructed by these methods in British Columbia and perhaps in Canada, has had two about 15 foot lifts of borrow material added to its height in the first two or three years of operation at a significant additional cost to the project. Campbell and Brawner (1971) cite an example of a large tailings dam where sand recovery appears to be of the order of 27% and sand shortages are anticipated in the first few years of operation. Galpin (1972) presents a case of another dam to be built by the downstream methods, where low sand recovery is anticipated due to the double cycloning specified for

the project.

Example II

Grain size distribution results on samples taken on July 8, 1973 at the Bethlehem tailings dam are presented in Table 7.3. It can be seen that the sand recovery from the first cyclone is about 30.5%. With the addition of Cyclone II, the sand recovery has been increased to an overall 50% - a significant 19.5% increase in the sand recovery.

At the Bethlehem site, the tailings are initially processed through a cyclone in the mill and only the coarse fraction remixed with a small amount of slimes is piped to the dam for further cycloning at the dam. It is estimated that only approximately 50% of the total tailings are transported to the dam. The rest is discharged directly into the pond from the mill.

The above 19.5% increase in the sand recovery, therefore, is on the basis of the material piped to the dam. For an average throughput rate of 17,000 tons per day for the mill, this increase in sand recovery could amount to about 1,000,000 cu yd of additional sand on a yearly basis. On a dam where sand supply might be critical, this order of increase in sand supply could amount to a saving of $500,000 in a single year at a conservative rate of $.50 per cu yd assuming that borrow materials would have been needed had the

sand supply not been adequate. It is recognized that there will be additional expenditures due to the second stage of cycloning. It is felt, however, that these expenditures are likely to be rather insignificant in view of the huge anticipated savings.

Example III

Grain size distribution curves are shown in Figure 7.7 for the Brenda materials as received in 1972. It can be seen that the tailings contain about 44% fines (-#200 material), the underflow sand about 8% and the overflow slimes about 72%. Under these circumstances, the sand recovery is about 44%. It is felt that by directing the above slimes through another stage of cyclones, the sand recovery could have been increased significantly, as shown in the previous examples, without compromising the quality of the sand.

More recent results (February, 1974) received from the site indicate that the sand recovery has now been increased to about 50%, partly through increased coarseness of the grind and partly through more efficient use of the cyclones. It is felt, however, that through the judicious use of cyclones, sand recovery can be increased further at this site.

It is interesting to speculate that if at a site the design requirements are such that sand with up to 20% fines is acceptable, for a coarse grind such as that shown in Figure 7.7, a sand recovery of about 67% can be expected provided sand losses in the slimes can be restricted to about 20%. That, for a production rate of 30,000 tons per day, would mean an increase of 1.5 million cu yd of sand supply for an increase in sand recovery from 50% to 67%. Furthermore there will be only two-thirds as much slimes to store.

It has been shown in the previous section that a significant percentage of fines is acceptable in the sand used for the construction of tailings dams to meet all possible requirements relevant to design and materials handling. For average conditions likely to exist at most tailings dams, fines to the extent of 20 to 25% may be acceptable in the sand with the overriding requirement that the sand placement technique should be such that no segregation of fines can occur. Furthermore, in this section, it has been demonstrated that through the judicious use of cyclones, a relatively high level of sand recovery can be achieved. This also has the effect of reducing the amount of slimes to be stored. For instance, for a typical tailings material with about 50% passing the #200 sieve, sand recovery of over 50% should be feasible. At the present time, for a design criterion of limiting the fines content to about

10% and a typical loss of sand sizes in the slimes of 30%, the sand recovery is usually limited to about 33%. It can be concluded, therefore, that the new methods presented here could very well be the answer to the problem of sand shortages so frequently experienced in the construction of tailings dams by the downstream methods.

7.4.2 Methods of Accelerating Consolidation of Slimes

The volume of slimes to be stored is another critical factor in the optimum design of a tailings disposal. Slimes are discharged into a pond in the form of a thin slurry. After the initial sedimentation which occurs rather rapidly, the slimes undergo consolidation which is a slow and long term process. The slimes will be consolidating during construction and for many years after the completion of a mining operation. The writer would envisage fairly deep lakes behind some of the large tailings dams in years to come.

It can be said that with increased consolidation much more solid material can be placed within the same initial depth. The height of a tailings dam required to provide storage space for the slimes is, therefore, a function of the degree of consolidation of slimes that can be achieved during construction. The higher the degree of consolidation of slimes during construction, the lower the dam height required to maintain crest level ahead of the pond elevation. There are various methods of accelerating the rate of

consolidation of hydraulically placed fine grained materials (see, for example, Bishop and Vaughan, 1972).

The case of double drainage (or underdrainage) is of particular interest here. It has been indicated in Chapter V that in the central portion of a tailings pond, consolidation occurs due to drainage of pore water in only the vertical direction. Pore pressure distribution curves are shown plotted in Figures 5.2B and 5.3B for a constant rate of deposition for the cases of impervious and pervious bases respectively according to Gibson (1958). Since the effect of consolidation can be best assessed in terms of increases in effective stress, the above curves are presented again in Figures 7.8 and 7.9 but this time, to illustrate effective stress distributions. It should be remembered that Figure 7.8 presents results in terms of excess pore pressures and Figure 7.9 in terms of total pore pressures. Effective stress at a certain depth in the deposit can be computed from the expressions given in the above figures. For instance at x/h = .5 for a deposit with the following parameters,

$h = 100$ ft

$\gamma_t = 95.0$ pcf,

$\gamma' = 32.5$ pcf, and

$\frac{m^2 t}{C_v} = 4.$

The effective stress for the impervious base is 420 psf, whereas in the case of the pervious base, the effective stress would be significantly larger, about 2400 psf. The significance of the differences in effective stress from these two cases can be best illustrated by an example.

Consider a typical deposit of slimes with the following properties:

$C_v = 125\ ft^2/yr = 3.7 \times 10^{-2}\ cm^2/sec$,

$m = 20$ ft/yr,

$h = 100$ ft, and

$t = 5$ yr.

The value of $\frac{m^2 t}{C_v}$ for the above case would be 16. It can be seen in Figure 7.8 that for an impervious base, the effective stress in the top 50% of the deposit is zero. In the lower half of the deposit, the average effective stress will be about 300 psf. In the case of a pervious base (Figure 7.9), the effective stress is zero only in the top 25% of the deposit. In the bottom 25% thickness of the deposit, the average effective stress is likely to be about 6000 psf and in the middle 50%, it would be about 1000 psf. On the basis of consolidation characteristics presented in Chapter III for the slimes, the net effect of all these changes in effective stresses is that 20% more solids can be stored in the case of a pervious base within the same depth of slimes. This difference is likely to be greater with increasing depth

of slimes. On the basis of the above discussion it would appear that in a typical high tailings dam, the depth of slimes may be only about 300 ft in the case of double drainage as compared to 400 ft for the impervious base.

On the basis of theoretical considerations it has been shown that a significant amount of reduction in dam height is possible with double drainage. In some cases bottom drainage may be feasible naturally due to a pervious foundation. In some other critical situations, installation of an underdrainage system or vertical drains (for example, sand drains) may be worth considering. In any event, field measurements in suitable situations would be of great assistance in obtaining better quantitative understanding of the possible effects of double drainage with respect to the point in question.

7.4.3 Stability of Upstream Slope

Another interesting question can be raised in view of the above discussion. If slimes are to undergo large amounts of post construction settlements, what effect would this have on the stability of the upstream slope of a tailings dam? In the case of a dam constructed by the centreline technique, the stability of the upstream slope would be very much dependent upon the lateral support of the adjoining slimes. Typically, however, the slimes adjacent to the sand-slime interface are likely to have a high degree of con-

solidation during construction due to lateral drainage into the sand. It is anticipated, therefore, that the post construction settlements would be rather small at the sand-slime interface and would gradually increase with increasing distance from this boundary. This point should be, nevertheless, thoroughly investigated in the case of a dam to be constructed by the centreline technique.

The above consideration would be a strong point in favour of maintaining a relatively high sand to slime permeability ratio for an unimpeded drainage. A sand to slime permeability ratio of 100 has been suggested in a previous section of this chapter. For the same reasons, an impervious seal against the upstream slope of an embankment to be constructed by the centreline technique would not be recommended.

In conclusion, it is to be emphasized that the stability of an upstream slope should be given due consideration. At the present time, the stability of an upstream slope is rather hastily passed over. It would be wise to remember that the failure of Barahona dam in Chile in 1928 was caused by the instability of the upstream slope (Dobry and Alvarez, 1967).

7.4.4 Modified Methods of Construction

It has been indicated previously in this thesis,

that both existing methods of constructing a tailings dam have some good points and some inherent weaknesses. Neither method is faultless.

The upstream method is economical but it has its limitations from the engineering point of view. It has been shown in a previous section of this chapter that a safe tailings dam can be built by this method following the basic principles of earth dam engineering and the proper procedures of materials handling. But the height to which such a dam can be built would be limited.

At the present time, for the particularly high tailings dams at the large low-grade open pit mines, downstream construction would generally be recommended. A dam can be built by this type of construction to any height and to satisfy all design reqirements. Construction by this method, however, is more costly than that by the previous method. Moreover, sand shortages in this type of construction, particularly in the early stages of dam building, can further add to the cost of the facility. Several methods have been discussed previously to alleviate this problem.

It appears that a desirable method might be a combination of the above two methods which will maintain at least some of the economy of the upstream construction and the engineering desirability of the downstream technique.

Two typical cases are selected for presentation purposes - one of a moderately high tailings dam for a medium size mining operation and another of a relatively high dam for a large open pit operation.

First consider the case of a moderately high tailings dam with a rate of pond build up of about 5 to 10 ft/yr. Typically, the dam should be built by the upstream method of construction as discussed in a previous section of this chapter. But the ultimate height of the dam is sufficiently large so that the upper portions of the dam are likely to be underlain by the previously deposited slimes near the base of the pond. It is assumed, however, that the beach length and the back ponding can be controlled so that no further inclusions of slimes can occur within the embankment sand.

It has been indicated that it would be feasible to maintain a minimum beach width of about 300 ft. In a typical set up, as shown in Figure 7.10, assume a starter dam of a height of about 30 ft. Subsequent dykes will be placed as required in a typical upstream construction until the dam reaches its ultimate height indicated by point A in Figure 7.10. In the above construction however, as dyke 8 is placed, the height of the dam reaches a critical stage in that the upper portion of the dam above this level will be underlain by previously deposited slimes if the upstream

method of construction is continued. From this stage onwards, it is suggested that construction should be no longer continued by the upstream technique; it should be carried out by the centreline technique instead and the ultimate crest of the dam will reach level B as shown in Figure 7.10.

It has been suggested in the above example that construction may proceed by the upstream technique until the dam reaches a height of about 110 ft. That would mean construction by the upstream technique during the first several years of operation while the sand supply might be critical. After this, the construction is to be carried out by the centreline technique but it is felt that at least the sand supply is not likely to be a problem at this later stage.

Now consider the case of a typical high tailings dam. For presentation purposes, it can be assumed that construction at this site would normally be carried out by the centreline technique. It is suggested that construction should commence in the usual way for the first few critical years. In the later stages of operation when sand yield begins to exceed the volume required for the normal centreline construction, it is suggested that further construction should be carried out by the downstream technique making sure that length of beach is maintained to at least upstream of the previous sand-slime interface of the centreline construction.

As the crest of the dam reaches point A on the downstream slope, it is suggested that all subsequent construction could be carried out by the upstream technique.

It is anticipated that in the above sequence, it might be possible to construct the top 100 ft or so of the dam by the upstream technique. In addition to the economy of the upstream construction in the later stages of the operation it is felt that this will perhaps allow the mining operator sufficient time to stabilize the downstream slope with erosion protection before the end of construction.

TABLE 7.1 Density of Hydraulic Sand Fills

Reference	Pertinent Comments
Casagrande, A. (1968A)	The relative density of hydraulic fills that were not artifically compacted is generally uniform and in the range of about 50%.
Whitman (1970)	Fills derived from fairly clean sands will generally have an average relative density of less than 60% (typically 45% to 60%) although wide variations will occur from point to point within the fill.
Schroeder and Byington (1972)	The results of studies of fill density for hydraulic fills above water indicate that a loose to medium dense fill can be expected with no compactive effort. Minimal amounts of compactive effort under proper conditions yield a medium dense fill.
Turnbull and Mansur (1973)	It appears that in a well-controlled hydraulic fill with less than 10% fines, a relative density of 50% to 60% can be obtained with no compaction.

TABLE 7.2 Density of Cycloned Tailings Sands

Dam	Fines Content (-#200 material) %	Relative Density Range	Relative Density Average	Reference	Comments
Bethlehem*	11 - 20 (reference Fig. 4.38)	45%-68%	55%	This Study	Measured by nuclear methods. Reference Figure 4.24.
Brenda**	6 - 12.5 (reference Fig. 4.38)	25%-100%	large portion over 50%	This Study	Measured by nuclear methods. Reference Figure 4.22.
Brenda**	3-6	4%-84%	47.0%	Klohn and Maartman (1973)	
Gibraltar*	10-12	33%-54%	44.5%	Klohn and Maartman (1973)	

NOTE - Maximum and minimum densities determined by methods similar to those recommended by ASTM (D-2049-69).

* On-dam cycloning
** Hydraulic cell construction

TABLE 7.3 Grain Size Distribution Results From Bethlehem Dam (Sampled July 8, 1973)

Cyclone	Sieve Size	Feed %	Underflow %	Overflow %
	+65	6.5	15.2	
	+100	14.6	29.2	9.9
I	+150	21.3	28.1	18.6
	+200	14.1	12.5	15.7
	+325	13.1	7.7	15.9
	-325	30.4	7.3	39.9
	Total	100.0	100.0	100.0
	+65		6.4	
	+100	9.9	27.4	1.3
	+150	18.6	36.3	11.0
II	+200	15.7	12.7	16.3
	+325	15.9	8.9	19.0
	-325	39.9	8.3	52.4
	Total	100.0	100.0	100.0

NOTES:

1. Overflow product from cyclone I is the feed for cyclone II.

	Cyclone I	Cyclone II	Overall
2. Sand Recovery	30.5%	28.5%	50.0%

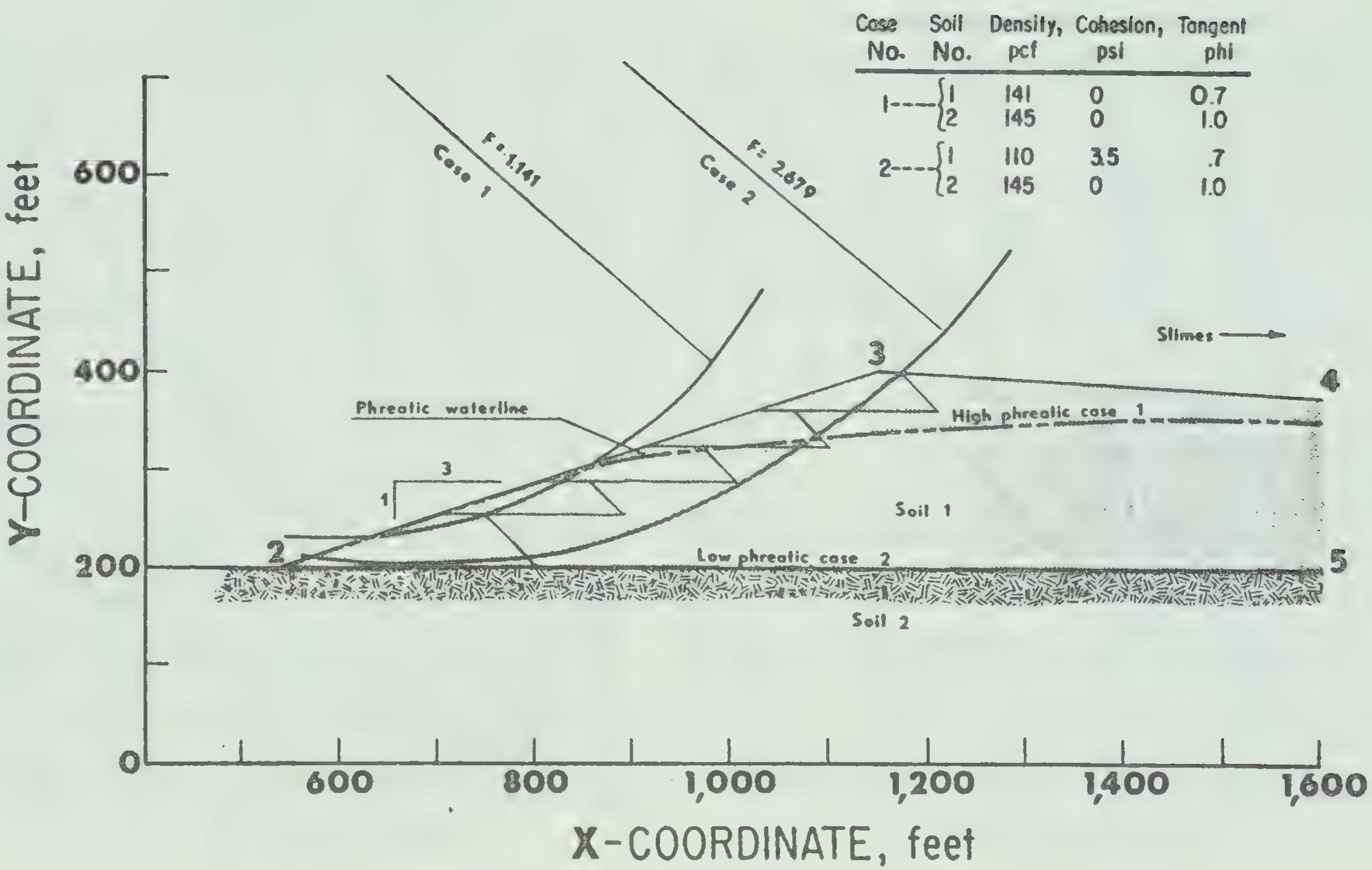


Figure 7.1 Results of Stability Analysis
(After Kealy and Soderberg, 1969)

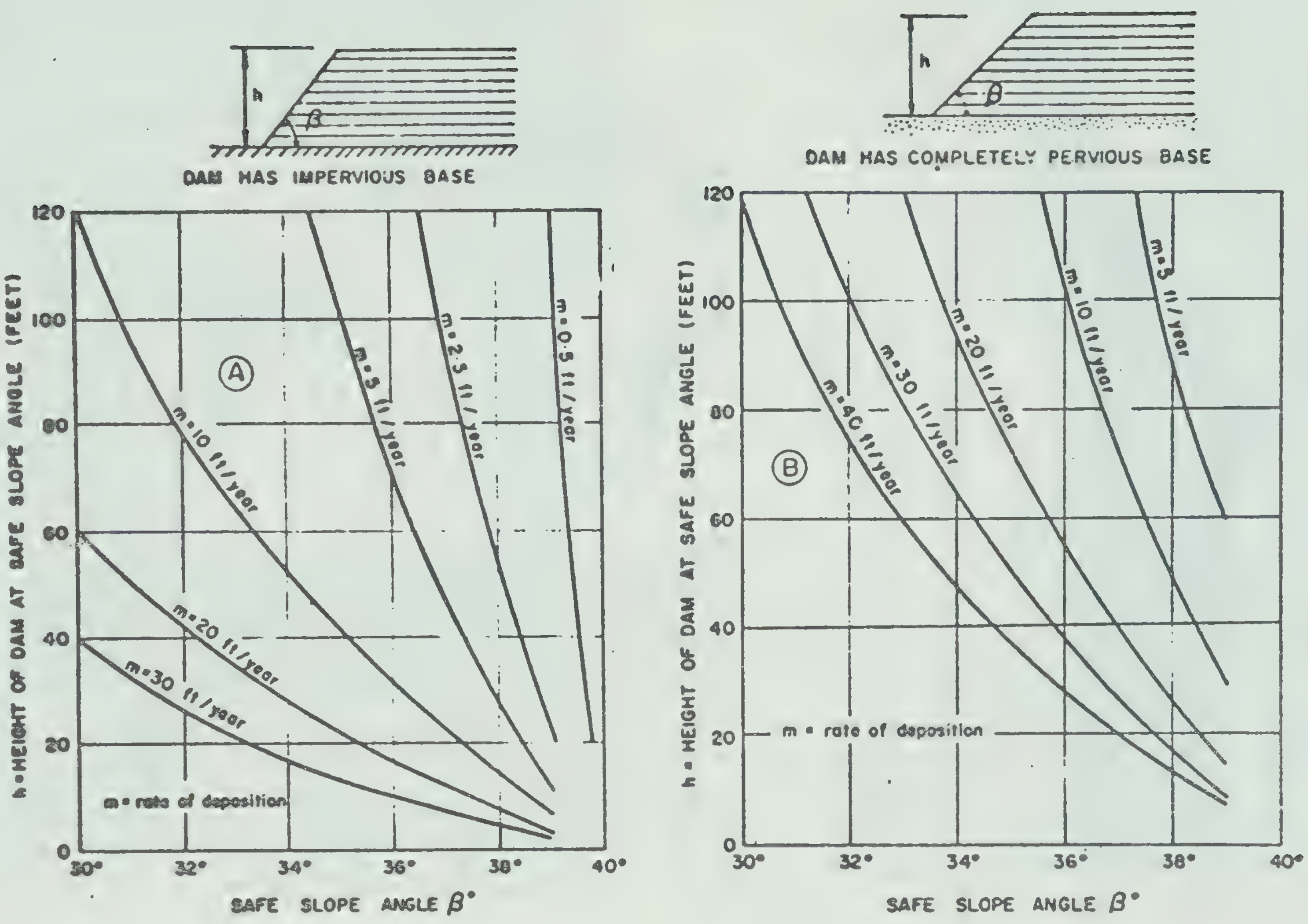
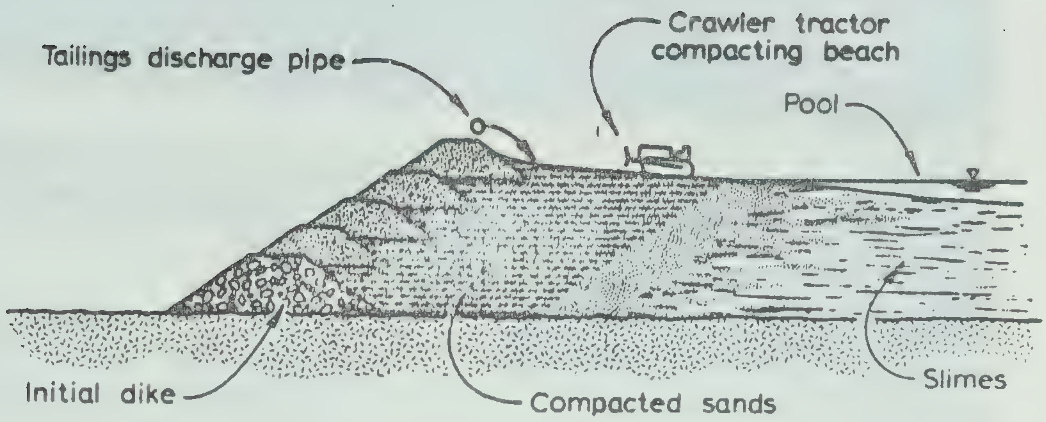
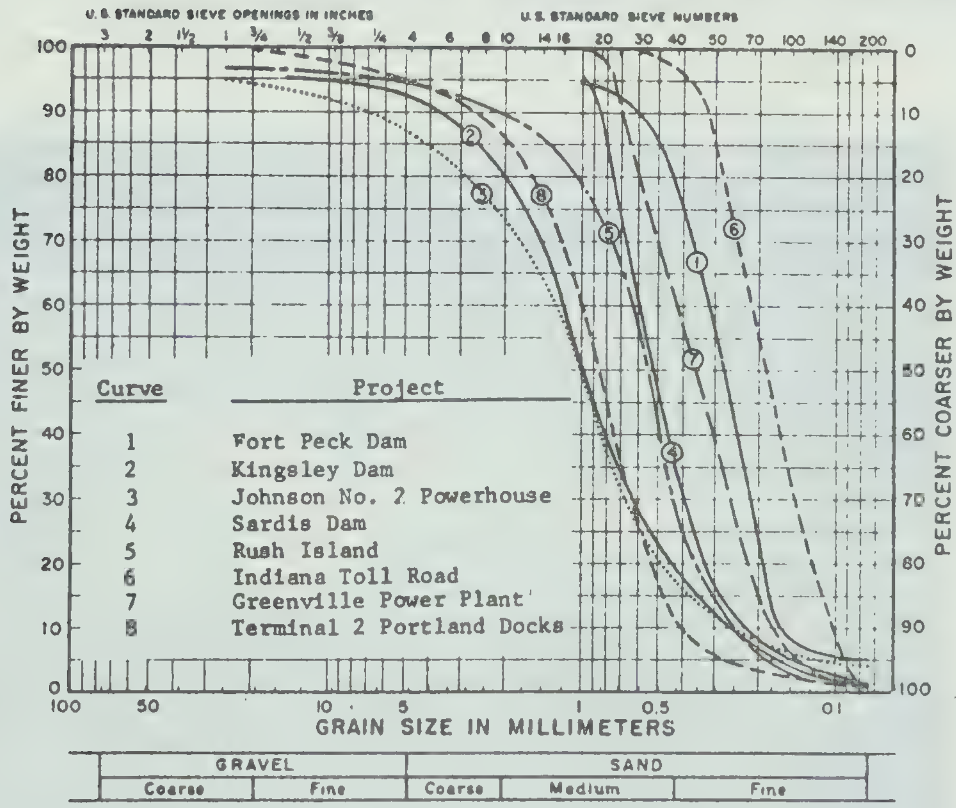


Figure 7.2 Design Graphs (After Blight, 1969)



(After Casagrande and MacIver, 1971)

Figure 7.2A A Safe Tailings Dam Being Constructed by the Upstream Method



(After Turnbull and Mansur, 1973)

Figure 7.3 Grain Size Curves For Hydraulic Sand Fills

Figure 7.4 Photograph illustrating water accumulation on otherwise clean sand during sand placement by hydraulic fill method.

Figure 7.5 Photograph illustrating a thin coating of fines left behind after water has evaporated.

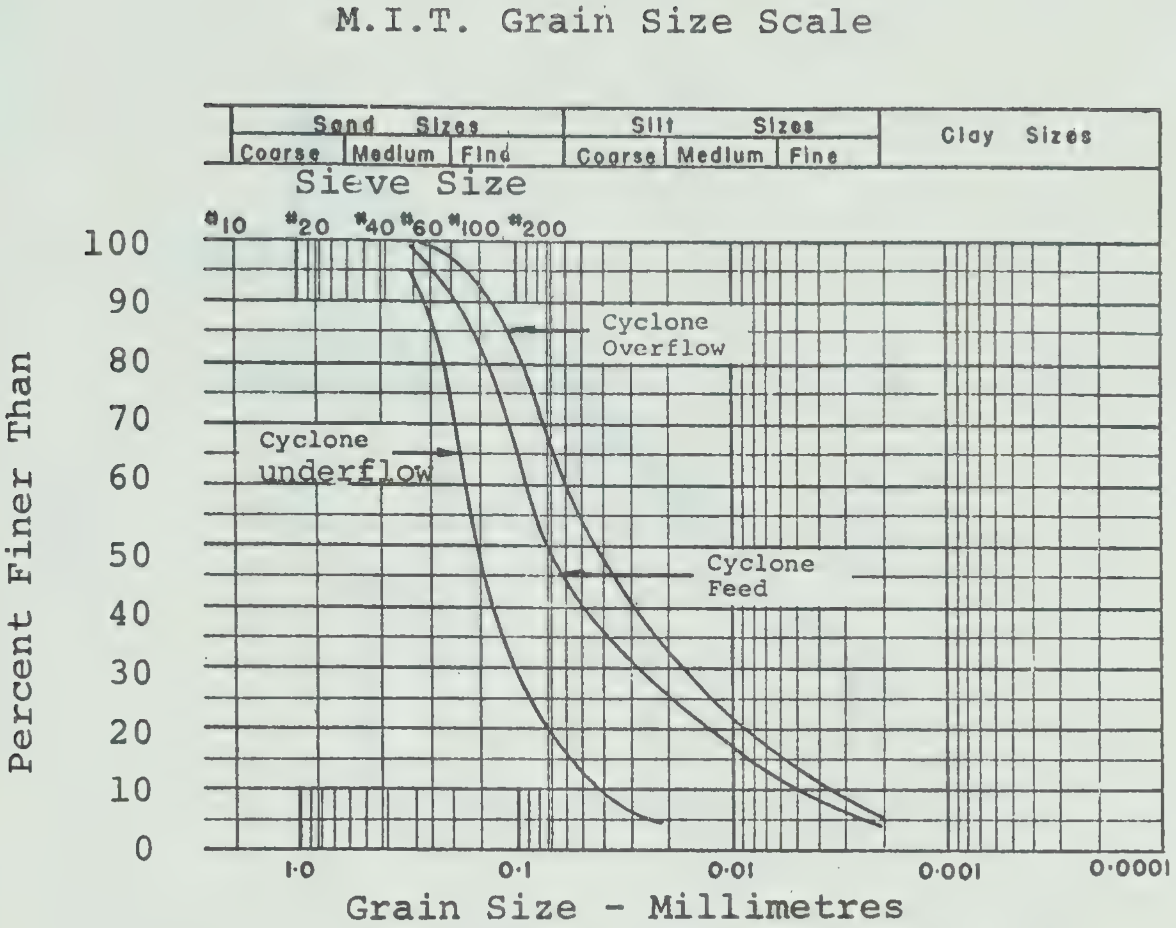


Figure 7.6 Grain Size Distribution Curves (reference Section 7.3.2.2 Example I)

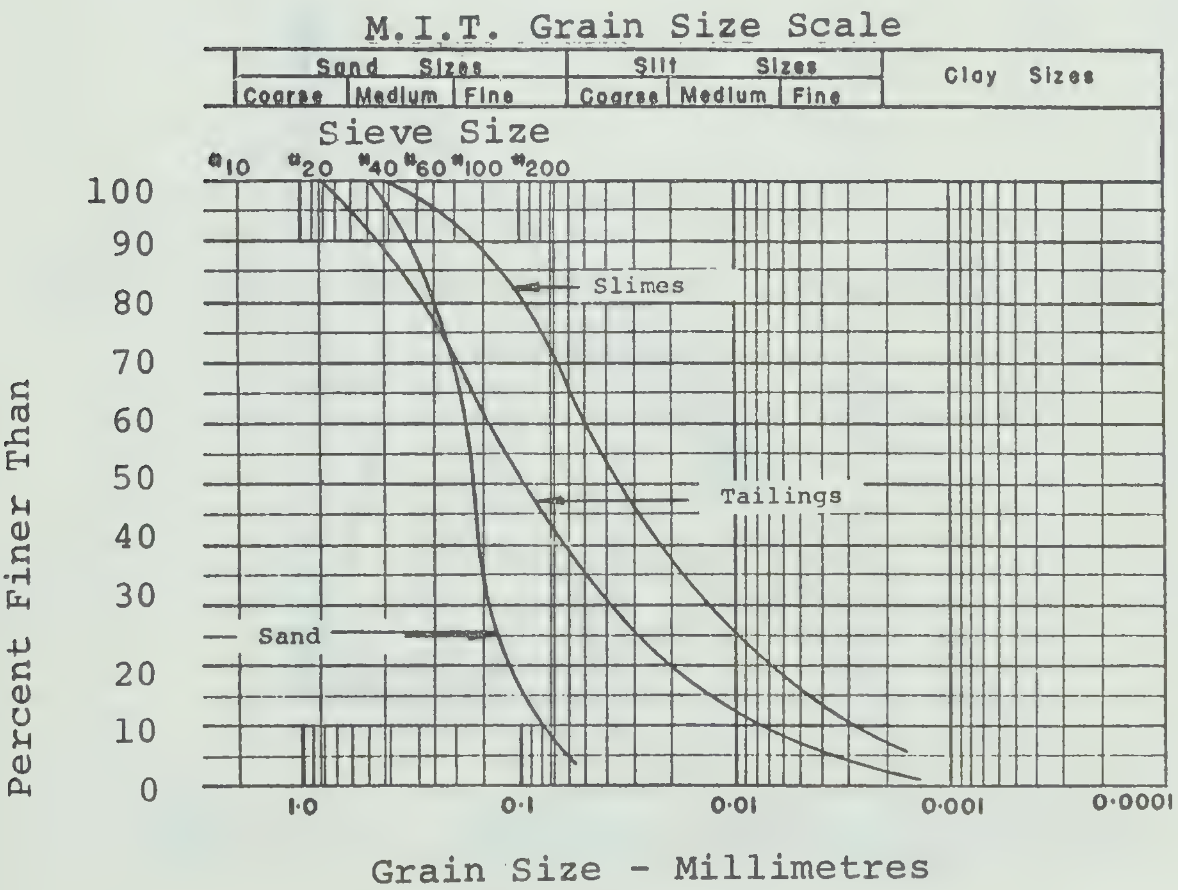


Figure 7.7 Grain Size Distribution Curves (reference Section 7.3.2.2 Example III)

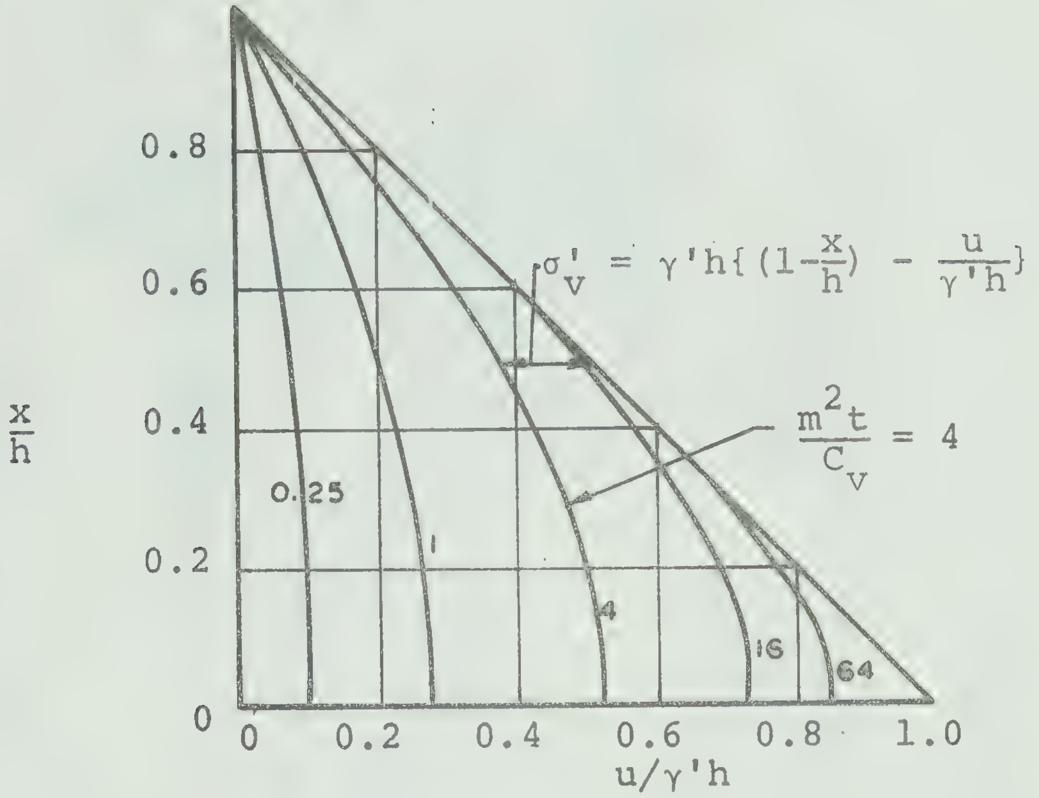


Figure 7.8 Effective Stress Distribution (Impervious Base; h = mt)

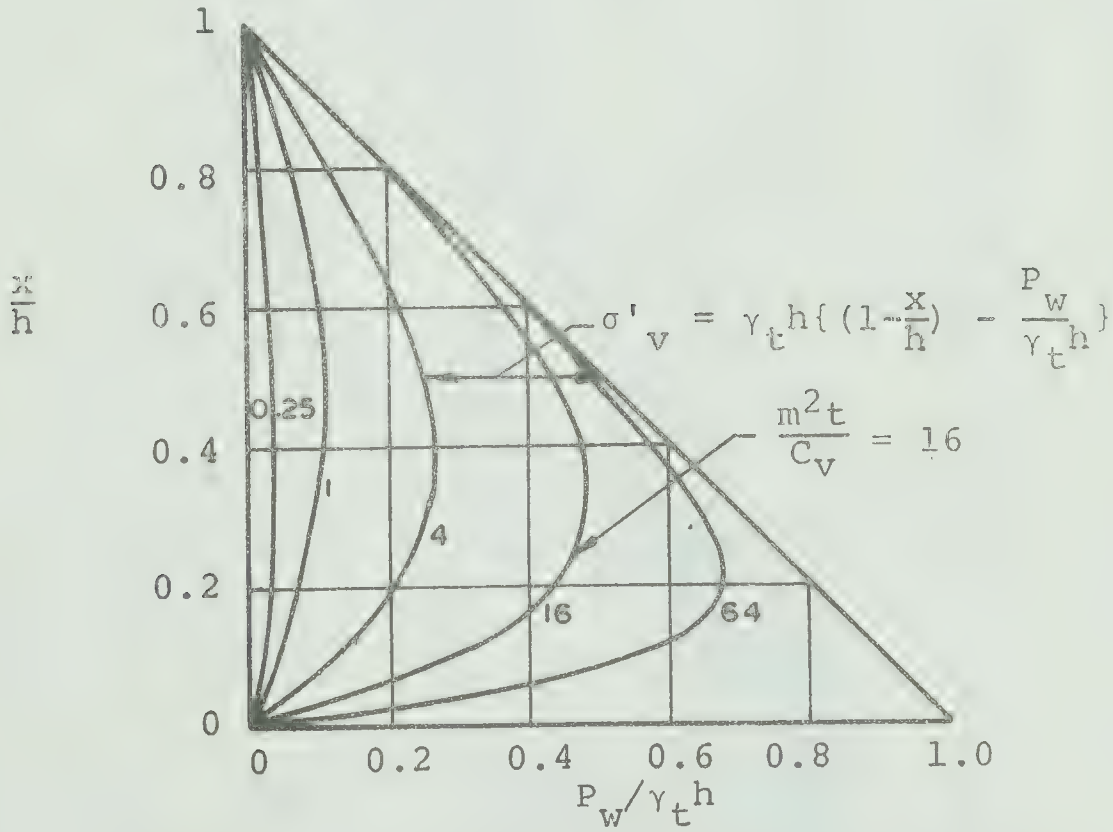


Figure 7.9 Effective Stress Distribution (Pervious Base; h = mt)

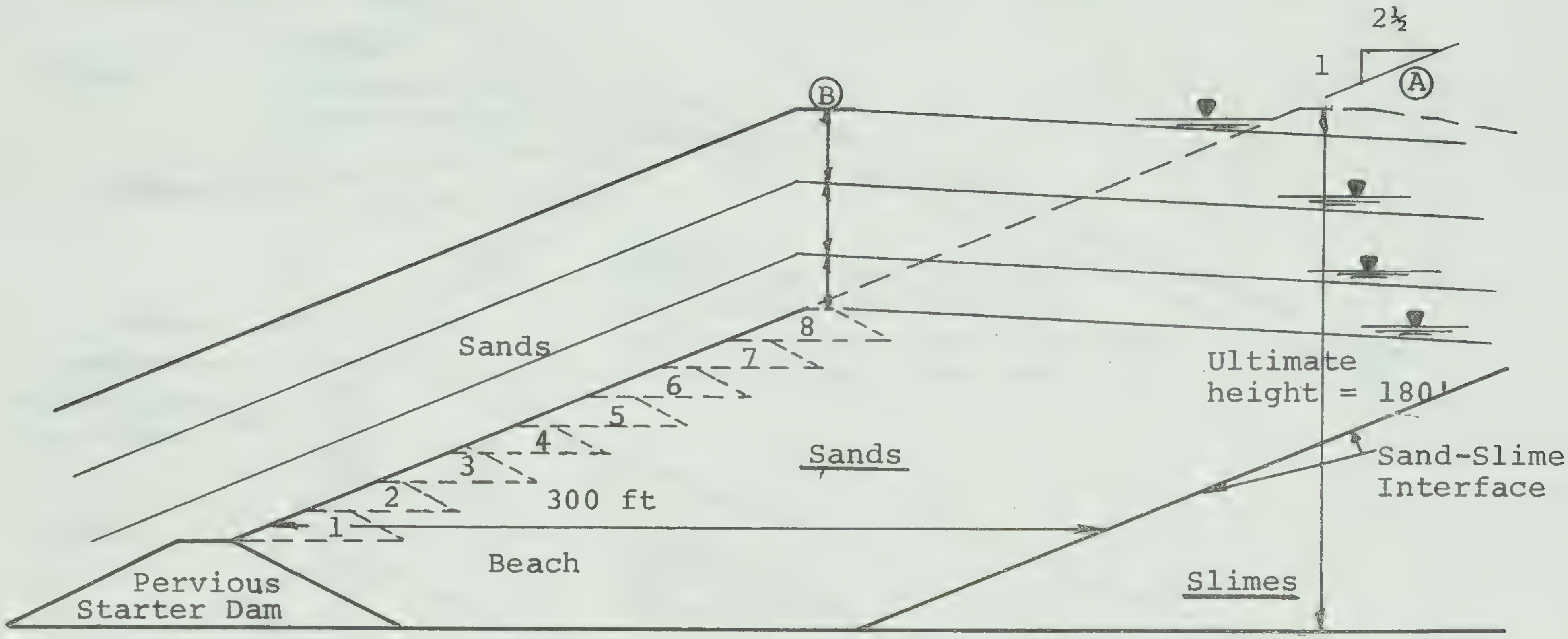


Figure 7.10 Modified Upstream Method of Construction

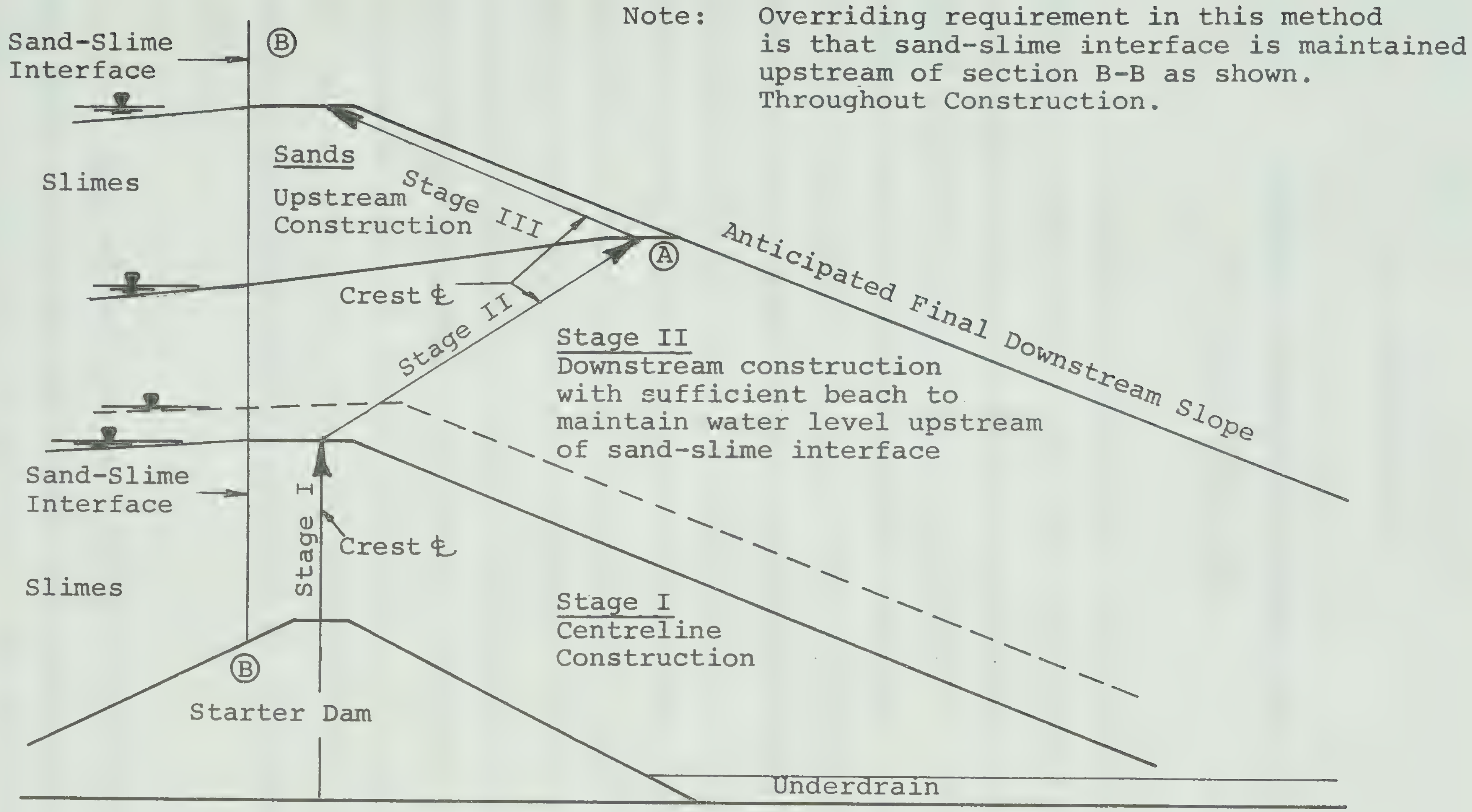


Figure 7.11 Modified Downstream Method of Construction

CHAPTER VIII

CONCLUDING REMARKS AND SUGGESTIONS FOR FURTHER RESEARCH

It has been shown in this thesis that tailings dams have been constructed, in the past, with little or no regard to even the most basic engineering principles which are commonly employed in the design of earth-fill dams. For instance, many tailings embankments have been constructed with side slopes too steep to provide an adequate factor of safety. Also, seepage control and bearing capacity of foundation materials have not been given due consideration in many cases. It can therefore be said that until recently, insufficient attention had been paid to the safety of tailings dams. With the growing concern for the protection of the environment and in view of the recent failures of waste embankments, notably the Aberfan disaster, the design of tailings dams has come under critical review in the last few years. It is now, generally recognized that the tailings embankments must be designed as safe engineering structures.

Most of the existing tailings embankments have been built by the upstream method using spigotting techniques for gravity separation of materials. The upstream method is

a simple technique which appears to produce economical construction. Careless construction by this method, however, has produced several unsafe tailings dams in the past. Proper procedures of materials handling and construction by this method have been discussed in this thesis. But, the height to which a dam can be built by this method would be limited. The methods of stability analysis for embankments to be constructed by this technique have been reviewed. It has been shown that the recently published work of Blight (1969) on the subject is in serious error.

For the high tailings dams at some of the large low-grade open pit mines, the newly evolved downstream methods of construction are generally recommended. This type of construction is more desirable from the engineering point of view but it is also more costly than that by the method noted previously. Modified methods of construction have been suggested to combine the economy of upstream method and the engineering desirability of the downstream techniques.

The single most critical factor which effects the cost of downstream construction is the amount of sand available for construction purposes. As in hydraulic fills, the fines content in the sand appears to be commonly restricted to a low value of less than about 12%. It has been shown in this thesis that a significantly higher percentage of fines is acceptable in the sand to meet all possible

requirements relevant to design and materials handling. For the average conditions likely to exist at most tailings dams, about 20 to 25% fines may be accepted in the sand with an overriding criterion that the sand placement technique should be such that no segregation of fines can occur. It has been further shown that through the judicious use of cyclones, a relatively high level of sand recovery can be achieved. In a typical case, for instance, it may be feasible to increase the sand recovery by as much as 50% by this method. Combined with the new high percentage of fines, this would completely change the situation with respect to sand supply at a site. Furthermore, this would reduce significantly the amount of slimes to be stored and hence, the height of dam required at any stage.

The rate of seepage flow through a tailings embankment is generally computed by a flow net analysis assuming a steady state seepage condition. In the fast rising ponds behind some of the large tailings dams, it has been found that the slimes are in an under-consolidated condition. The rate of seepage flow is, therefore, controlled by excess pore pressures in the slimes. A suitable formulation and an analytical solution is presented for computation of seepage flow in the above case. It has been further indicated that an impervious seal against the upstream slope may be only necessary under special circumstances. Furthermore, in the case of centreline construction, an impervious seal may

have a detrimental effect on the stability of the upstream slope.

It has been shown by theoretical considerations that double drainage can accelerate consolidation of slimes during construction and can have appreciable effect on the height of dam required at a site. Further work, however, is necessary in this area, particularly with respect to field observations.

In addition to the usual monitoring of seepage control by piezometer installations, field measurements of in situ density and permeability are needed to provide a flexible approach to the final optimum design of a tailings dam. Existing methods of measuring these parameters are shown to be unsatisfactory in the case of tailings dams. New methods have been explored, and suitable equipment and test techniques developed. In situ density is found using a nuclear probe driven into the embankment and in situ permeability above the water table is deduced from a constant head infiltration test. Both tests have been applied under field conditions and shown to give reliable results.

On the basis of limited field data presented in this thesis, it can be tentatively concluded that on-the-dam cycloning produces fill densities of 45 to 55% relative density. These results are quite comparable to those

generally reported for hydraulic fills. Further field measurements are, however, needed before firm conclusions can be drawn.

A general review of earthquake effects on the stability of tailings dams has been presented in this thesis. On the basis of a review of reported case histories and the various criteria generally recommended, it has been concluded that a relative density of 50 - 60% should be sufficient to preclude liquefaction in the case of tailings dams where design ground accelerations are not likely to exceed 0.1g. Higher densities will be needed in areas of higher seismicity. It has been indicated that earthquake induced settlements and shear displacements in the case of drained tailings embankments are likely to be of insignificant magnitude and, therefore, are not an issue in an otherwise well designed tailings dam.

It should be noted that most of the reported cases of earthquake related experiences have been with respect to natural materials. Furthermore, most of the reported research testing with respect to liquefaction studies has been performed on uniform and clean sands, generally devoid of fines. Research testing of actual tailings sands to simulate earthquake effects is, therefore, needed.

Another issue that deserves attention is the

criterion for compaction control. A minimum relative density is frequently used for compaction control in the case of hydraulic fills of fairly clean sand (fines content less than about 10%). For a higher fines content, a criterion of comparative compaction (for example, percentage of Standard Proctor Density) is often recommended. In the construction of tailings dams, if a high percentage of fines is to be accepted in the sand as suggested in this thesis, a further clarification of this issue is needed.

Finally, it must be emphasized that performance data from tailings dams at the present time is rather scarce, to say the least. Various new techniques and suggestions have been put forth in this thesis which should improve the state-of-the-art of designing tailings dams. Case histories to record relevant performance data, however, are badly needed.

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APPENDIX A

GUIDELINES FOR THE DESIGN, CONSTRUCTION AND OPERATION OF TAILINGS IMPOUNDMENTS IN BRITISH COLUMBIA

An engineering report can be required under the authority of Section 7 (3) of the Mines Regulation Act or Section 6 (3) of the Coal Mines Regulation Act to allow a determination to be made of the stability of a tailings impoundment. The engineering report should be signed by a professional engineer specializing in the field of soil mechanics and should contain the following: -

1. <u>Maps and Drawings</u>

   (a) The map or maps should show the location of the impoundment or dam, the physical features of the downstream area which might be affected by any dam failure, and the upstream watershed draining to the dam structure.

   (b) The drawings should show the layout in plan; typical cross-sections of all embankments, and if applicable, anticipated future extensions; and the location and design of the spillway or diversionary drainage to be installed at the termination of mining operations or on any prolonged shutdown. Embankment design should include information on heights, top width, side slopes, freeboard, seepage control, and protection of the embankment surfaces.

2. <u>Design Analysis</u>

An analysis of the integrity of the proposed design. The information should include: -

   (a) Results of geologic studies and site investigation including logs of drill holes, field permeability tests and ground water levels.

   (b) Results of soil tests on foundation materials including shear strength and consolidation test data.

   (c) Description of the engineering properties of the materials to be used for construction of the dam.

   (d) Results of stability studies to assess:
   - Foundation stability and settlement
   - Slope stability
   - Surface erosion control
   - Seepage and piping
   - Earthquake stability

3. Hydrology

A hydrological assessment of the foundations and of the structure is required, based on the location of the impoundment. This should include surface and subsurface water conditions, diversion of watercourses, drainage and runoff from the upstream basin, and erosion control. Also included should be an assessment of the seepage through the foundation or dam itself.

4. Construction

An operating manual should be drawn up to show construction specifications, rate of construction, and control due to weather conditions. The supervision must be detailed to ensure that there is no possibility of error being made during site preparation and construction of the dam. In particular the compaction requirements must be set out to make certain the fill material is placed at or near optimum water content and in this regard details are required of the drying method or water addition. Density standards are necessary and in situ density tests of the fill should be outlined.

5. Operating Control

Adequate instrumentation and monitoring of ground water and of seepage water should be provided. Any possible settling or movement in the foundation or the dam structure should be measured. Trained and adequate supervision to ensure the construction and operation of the impoundment is carried out to specifications must be provided.

APPENDIX B

EQUIPMENT DETAILS

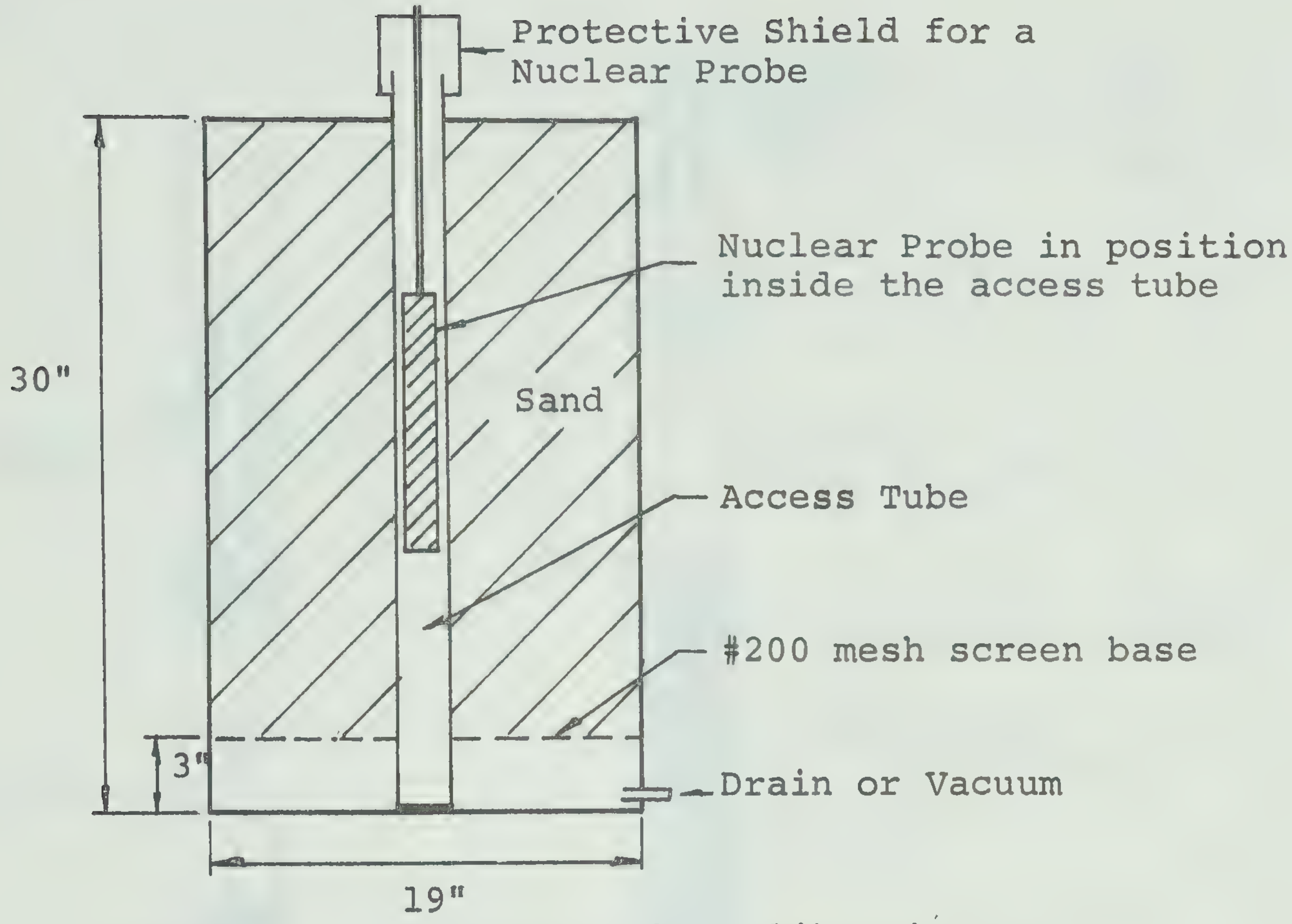


Figure B.1 Nuclear Probe Calibration Set Up

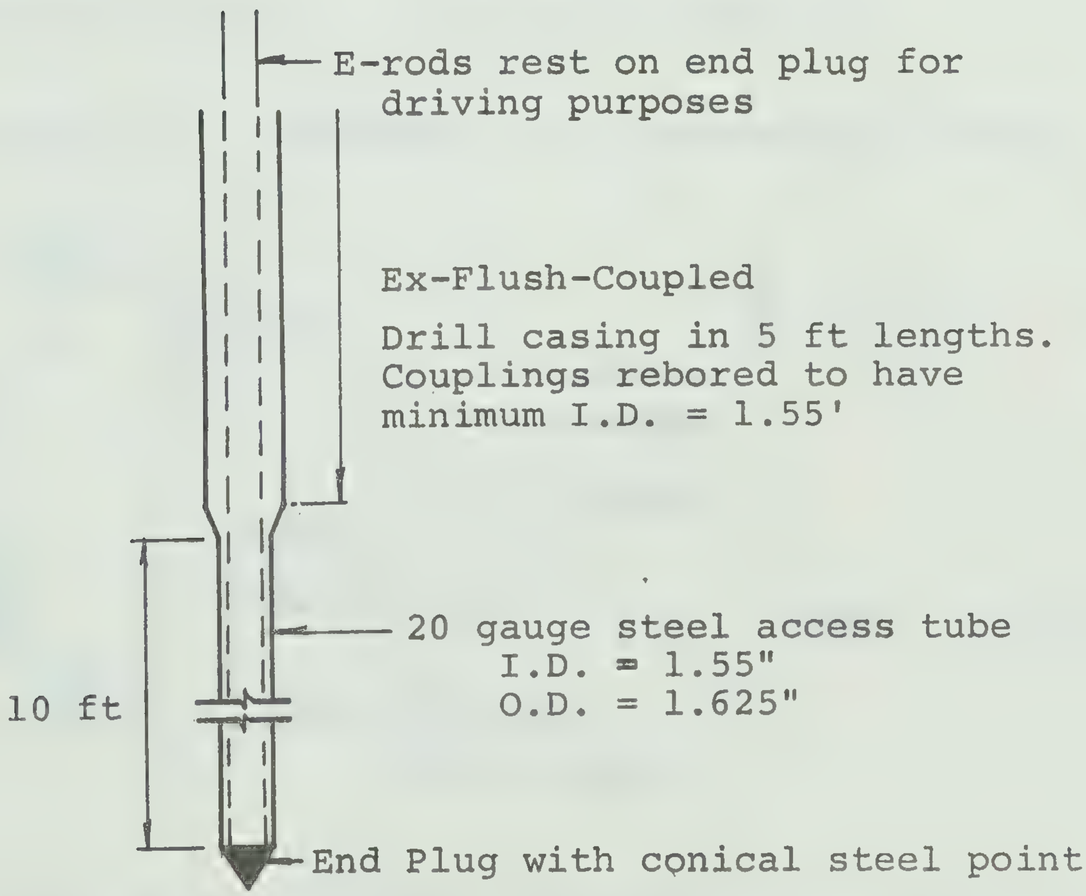
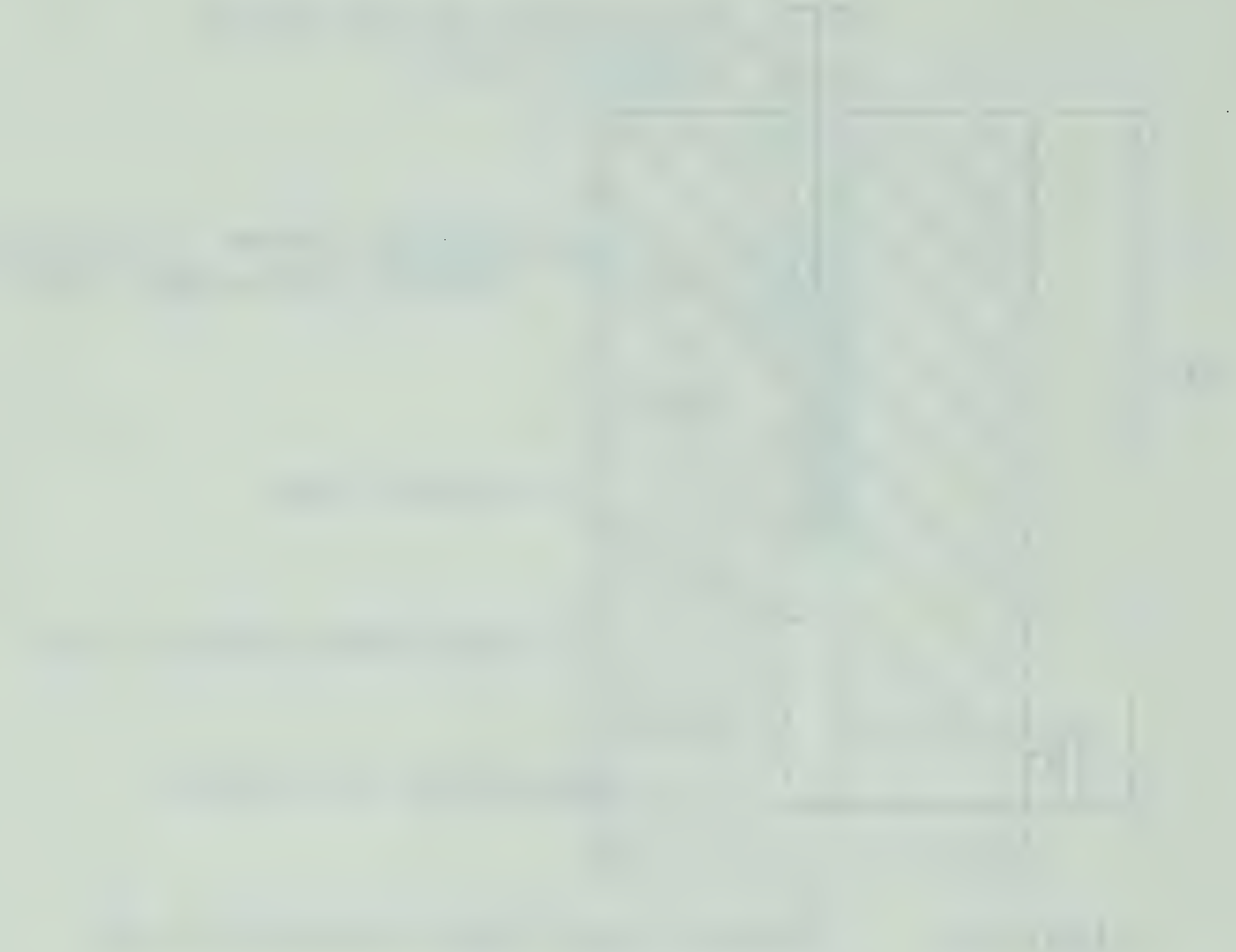


Figure B.2 Access Tube and Extensions



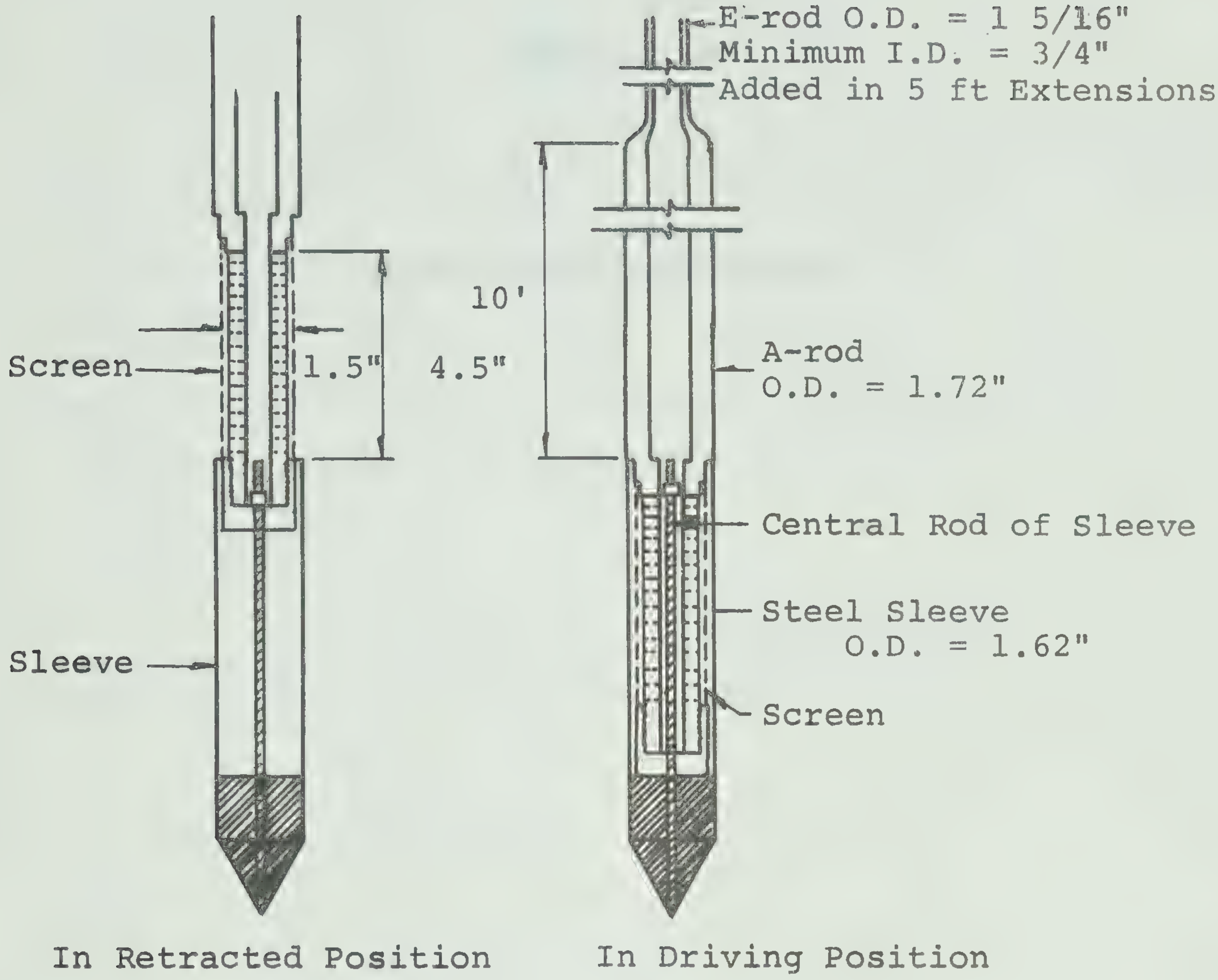


Figure B.3 Piezometer with Retractable Sleeve

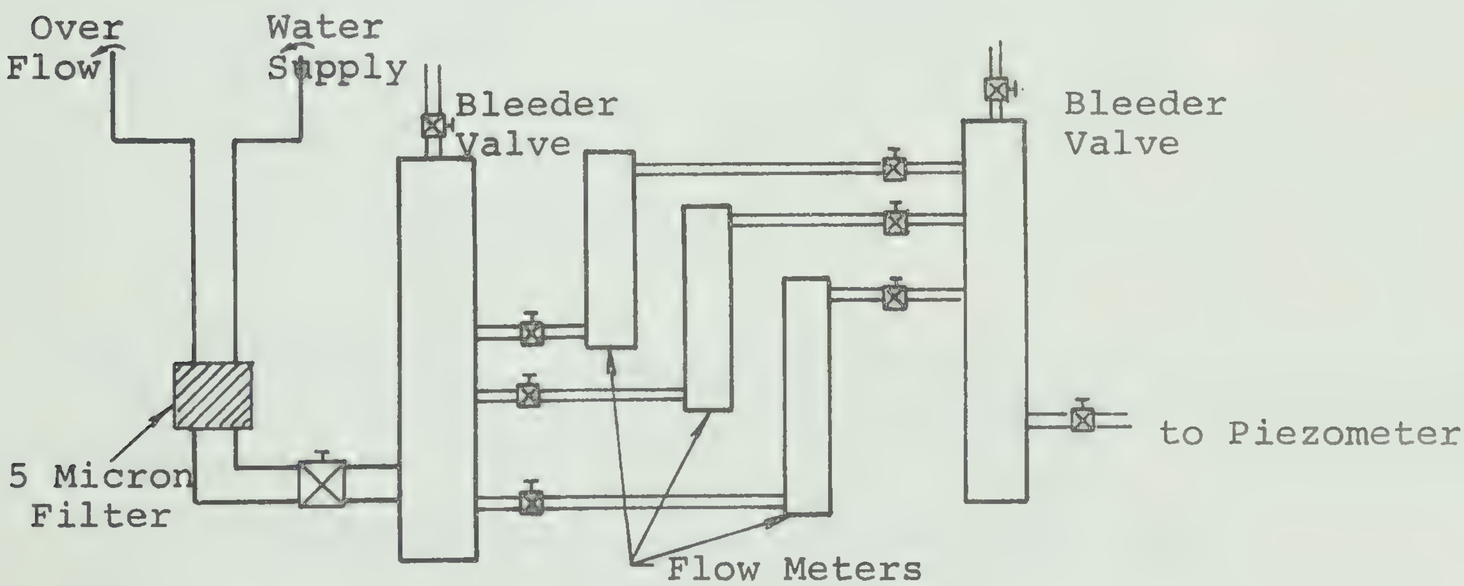


Figure B.4 Constant Head Permeability Apparatus

APPENDIX C

FLOW FROM A POINT SOURCE

FLOW FROM A POINT SOURCE

Discharge from a point source into an otherwise dry soil is an axi-symmetrical seepage problem for which an approximate analytical solution has been given by Palubarinova-Kochina (1962). Because this reference gives only a brief statement of results from an earlier unobtainable Russian paper, details of the derivation are given here.

A potential function Φ and the stream function Ψ are defined with respect to velocity components v_z and v_r by

$$v_z = \frac{\partial \Phi}{\partial z} = -\frac{1}{r}\frac{\partial \Psi}{\partial r} \qquad \text{.....(C.1a)}$$

and

$$v_r = \frac{\partial \Phi}{\partial r} = \frac{1}{r}\frac{\partial \Psi}{\partial x} \qquad \text{.....(C.1b)}$$

Φ is related to the piezometric head h and the pressure by,

$$\Phi = - kh = -k\cdot(\frac{p}{\gamma_w} - z) \qquad \text{.....(C.2)}$$

since the z axis is positive downward as shown in Figure 4.27.

The potential and stream functions are:

$$\Phi = - \frac{kq}{\sqrt{z^2 + r^2}} + \frac{kq_1}{\sqrt{(z+b)^2 + r^2}} + kz \qquad \text{....(C.3)}$$

$$\Psi = \frac{kr^2}{2} - \frac{kqz}{\sqrt{z^2 + r^2}} + \frac{kq_1(z+b)}{\sqrt{(z+b)^2 + r^2}} \qquad .....(C.4)$$

In (C.3) and C.4) positive values of the square roots apply. This affects the signs of terms used below.

The resulting velocity components, derivable from either (C.3) or (C.4) are,

$$v_z = \frac{kqz}{(z^2 + r^2)^{3/2}} - \frac{Kq_1(z+b)}{\{(z+b)^2 + r^2\}^{3/2}} + k \qquad ....(C.5a)$$

$$v_r = \frac{kqr}{(z^2+r^2)^{3/2}} - \frac{kq_1}{\{(z+b)^2 + r^2\}^{3/2}} \qquad .....(C.5b)$$

The potential function represents a source at the origin of strength

q = 4 k¶q,

a sink on the z axis at z = -b of strength

Q_1 = -4¶ kq_1,

and a uniform field with velocity parallel to the positive z axis. The "free" streamline is ABC (Figure 4.28).

Let the stagnation point A (Figure 4.28), be at z = -a, then from (C.4)

$$\Psi_o = \frac{kqa}{\sqrt{a^2}} + \frac{kq_1(b-a)}{\sqrt{(b-a)^2}}$$

or

$$\Psi_o = k(q + q_1) \qquad \text{.....(C.6)}$$

is the value of Ψ on the free streamline. The equation of this streamline is,

$$q + q_1 = \frac{r^2}{2} - \frac{qz}{\sqrt{z^2 + r^2}} + \frac{q_1(z+b)}{\sqrt{(z+b)^2 + r^2}}$$

or

$$q + q_1 = \frac{r^2}{2} - \frac{q}{\sqrt{1+(\frac{r}{z})^2}} + \frac{q_1}{\sqrt{1 + (\frac{r}{z+b})^2}} \qquad \text{.....(C.7)}$$

As $z \to \infty$, the half width of the region of flow becomes,

$$q + q_1 = \frac{r^2_\infty}{2} - q+q_1$$

$$r_\infty = 2\sqrt{q} \qquad \text{.....(C.8)}$$

On the r axis, i.e. at $z = 0$,

$$q + q_1 = \frac{r^2_o}{2} + \frac{q_1 b}{\sqrt{b^2 + r_o^2}} \qquad \text{.....(C.9)}$$

At $z = 0$, $r = r_o$, the zero pressure condition is

$$\Phi - kz = 0 \qquad \text{.....(C.10)}$$

so from (C.3)

$$-\frac{q}{r_o} + \frac{q_1}{\sqrt{b^2 + r_o^2}} = 0$$

from which

$$r_o^2 = \frac{q^2 b^2}{q_1^2 - q^2} \qquad \text{.....(C.11)}$$

Now substituting for r_o^2 in (C.9)

$$q + q_1 = \frac{q^2 b^2}{2(q_1^2 - q^2)} + \frac{q_1 b}{\sqrt{b^2 + \dfrac{q^2 b^2}{q_1^2 - q^2}}}$$

from which

$$b^2 = \frac{2(q+q_1)(q_1^2 - q^2)}{q^2} - 2\frac{(q_1^2 - q^2)^{3/2}}{q^2} \qquad \text{.....(C.12)}$$

On the z axis at $z = -a$, $v_z = 0$, so from (C.5a)

$$-\frac{q}{a^2} - \frac{q_1}{(b-a)^2} + 1 = 0$$

or

$$a^2(b-a)^2 - q(b-a)^2 - q_1 a^2 = 0 \qquad \text{.....(C.13)}$$

Finally, for zero pressure at the stagnation point,

$$\Phi + ka = 0$$

so from (C.3)

$$-\frac{q}{a} + \frac{q_1}{(b-a)} = 0$$

or

$$a = \frac{qb}{q + q_1} \qquad .....(C.14)$$

Now a, b anc q_1 can be found from equations (C.12), (C.13) and (C.14).

From (C.14),

$$b - a = \frac{q_1 b}{q + q_1}$$

Substituting into (C.13),

$$\frac{q^2b^2}{(q + q_1)^2} \cdot \frac{{q_1}^2b^2}{(q + q_1)^2} - \frac{q{q_1}^2b^2}{(q + q_1)^2} - \frac{q^2q_1b^2}{(q + q_1)^2} = 0$$

$$qq_1b^2 = (q + q_1)^3$$

$$b^2 = \frac{(q + q_1)^3}{qq_1} \qquad .....(C.15)$$

Equating (C.12) to (C.15) gives

$$\frac{2(q+q_1)(q_1{}^2-q^2)}{q^2} - \frac{2(q_1{}^2-q^2)^{3/2}}{q^2} - \frac{(q+q_1)^3}{qq_1} = 0$$

Now let,

$$\alpha = q_1/q \qquad \text{.....(C.16)}$$

so the above becomes

$$2\alpha(\alpha+1)(\alpha^2-1) - 2\alpha(\alpha^2-1)^{3/2} - (\alpha+1)^3 = 0$$

$$2(\alpha^2-1)^{3/2} - 2\alpha^4 - 2\alpha^3 + 2\alpha^2 + 2\alpha + \alpha^3 + 3\alpha^2 + 3\alpha$$
$$+1 = 0 .$$

$$2\alpha(\alpha^2-1)^{3/2} - 2\alpha^4 - \alpha^3 + 5\alpha^2 + 5\alpha + 1 = 0 \quad \text{...(C.17)}$$

This has the solution,

$$\alpha = 3.54870 \qquad \text{.....(C.18)}$$

Thus

$$b^2 = \frac{(\alpha + 1)^3}{\alpha} q \qquad \text{.....(C.19a)}$$

$$b = 5.14987 \sqrt{q} \qquad \text{.....(C.19b)}$$

and $$a = \left(\frac{1 + \alpha}{\alpha}\right)^{\frac{1}{2}} \sqrt{q}$$

$$a = 1.13216 \sqrt{q} \qquad \text{.....(C.20a)}$$

or $$a = 0.31938 \sqrt{Q/k}, \qquad \text{.....(C.20b)}$$

and $$r_o^2 = \frac{b^2}{\alpha^2 - 1}$$

$$r_o = \frac{(\alpha + 1)^{3/2}}{\alpha^{\frac{1}{2}} (\alpha^2 - 1)^{\frac{1}{2}}} \sqrt{q}$$

$$r_o = 1.51249 \sqrt{q} \qquad \text{.....(C.21)}$$

Since $r_\infty = 2\sqrt{q}$, equation (C.8),

$$r_\infty = \frac{2}{1.13216} a = 1.7665a \qquad \text{.....(C.22)}$$

Therefore the potential and stream functions are:

$$\Phi = Ka \{- \frac{0.7802}{\sqrt{Z^2 + R^2}} + \frac{2.7686}{\sqrt{(Z+4.5)^2 + R^2}} + Z\} \text{....(C.23a)}$$

$$\Psi = Ka^2 \{\frac{R^2}{2} - \frac{0.7802}{\sqrt{Z^2 + R^2}} + \frac{2.7686\ (Z+B)}{\sqrt{(Z+4.5)^2 + R^2}}\} \text{....(C.23b)}$$

where,

$$\frac{q}{a^2} = 0.7802 \qquad \text{.....(C.24a)}$$

$$\frac{q_1}{a^2} = 2.7686 \qquad \text{.....(C.24b)}$$

$$Z = z/a \qquad \text{......(C.24c)}$$

$$R = r/a \qquad \text{......(C.24d)}$$